THE CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

WALKER BROOK FLOOD PLAIN STUDY - PHASE II

Restigouche County

REPORT

April, 1992

Fenco Engineers Inc.

In association with

MacLaren Plansearch Inc.

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92-04877/

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WALKER BROOK FLOOD PLAIN STUDY, PHASE II

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1.0 LETTER OF TRANSMITTAL

April 27, 1992

Mr. J. G. Lockhart, P. Eng.
Program Manager
Canada - New Brunswick
Flood Dama[illegible]eduction Program
Department of Environment
P.O. Box 6000
Fredericton, N.B.
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Dear Mr. Lockhart:

WALKER BROOK FLOOD PLAIN STUDY - PHASE II
YOUR FILE 6615-7

We are pleased to forward fifty (50) copies of the above-captioned study along with one copy of Technical Appendices. As you are aware, this study dragged on due to minor unresolved matters between ourselves and comments made by reviewers, and in the end some compromises were made so the basic study results would not be lost for the next stage of possible structural flood control installation in the Walker Brook Flood Plain Area.

This final submission was then further delayed due to pressure of other assignments and we do apologize for that lost time.

In view of its long history of flooding, the Walker Brook floodplain was given high priority under both the Flood Risk Mapping and Studies Agreements in 1976. A multi-phased project was designed in the early 1980s. Phase I of the project, the identification of flood risk, was completed in March of 1986 when the Ministers of Environment for Canada and New Brunswick designated, on an interim basis, the floodplain along Walker Brook from just north of the Route 11/Route 270 interchange to its mouth. A series of nine 1:1 200 scale engineering maps, and a multi-coloured information map, were prepared.

This study constitutes Phase II of the project; an engineering, economic and environmental evaluation of structural and non-structural measures which reduce flood damages over the long term. This report summarizes the findings of a technical report prepared on the subject, focussing on the remedial works evaluation. The reader is referred to the technical report for further details.

The principle cause of flooding along Walker Brook is inadequate capacity of stream crossings, or the inability of these structures to pass ice, debris and sediment. Inadequate drainage of local runoff compounds this problem. Solutions to the flooding problem must take these factors into consideration. Alternatives which are designed to control open-water flooding may not be totally effective in preventing flood damages, especially in locations affected by backup from culverts which tend to block with ice, debris and sediment.

Flooding in the area of the shopping mall in Atholville and in the St. Albert area of Campbellton is primarily caused by floodwaters which (a) spill out of Walker Brook upstream of the Route 11/Route 270 interchange, and (b) runoff from the hills on the westerly side of the brook.

This study examines six structural options focussed on the St. Albert area which is primarily affected by out-of-channel spills upstream of the Route 11/Route 270 interchange. This spill also affects the shopping mall and surrounding area upstream of Dover Street. These are summarized as follows:

SUMMARY OF STRUCTURAL FLOOD CONTROL OPTIONS

Option	Description	Capital Cost
1	Berm, Floodwall and Culvert Enlargement on Route 270 above the Route 11 Interchange	$ 40 000
2	Option 1, Drainage Improvements - Route 11 Interchange to Alexander Street	$ 265 000
3	Option 2, Diversion Structure, Diversion Channel along Route 11, and culverts under Route 11 above Gallant Drive	$ 900 000
4	Option 2, Berms upstream of each crossing, replacement of Dover Street (upper) culvert with open-span bridge, enlargement of Alexander Street footbridge, and associated channelization	$1 200 000
5	Option 2, Berms upstream of Dover Street (lower crossing), replacement of culverts at Pine and Dover Streets (upper crossing) with open-span bridges. Enlargement of Alexander Street footbridge and associated channelization and provision for an additional culvert at Route 11.	$1 600 000
6	Option 2, Berms upstream of Pine and Dover (lower crossing) Streets	$ 400 000

These are fully reviewed in Chapter 6.

There remains considerable undeveloped flood-prone lands in the Walker Brook Basin. The City of Campbellton and the Village of Atholville are encouraged to restrict development in the designated floodway, and to insist on floodproofing structures in the remainder of the flood risk area. Failure to do so will lead to an exacerbation of the problem, not only for those who build there, but to those already there, as flood levels will increase as the floodplain is infilled with development.

Respectively submitted,

FENCO Engineers Inc.

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2.0 INTRODUCTION

2.1 GENERAL FRAMEWORK OF THE STUDY

The initial phase of the Walker Brook Flood Plain Study, Defining the Extent of the Flood Problem and Designation, was carried out by Spear, MacDonald & Associates Limited in association with Cumming-Cockburn & Associates Limited (hereinafter referenced as SMCC). The Phase I work was performed over the March 1982 to March 1986 period.

This Phase II of the Project was commissioned January 11, 1988, by the Steering Committee of the Canada-New Brunswick Flood Damage Reduction Program. The study has been carried out by FENCO Engineers Inc., a division of LAVALIN Inc., assisted by associate division MacLaren Plansearch Inc. in areas of hydrologic analyses and hydraulic modelling, and by Mr. Ian D. Thomas, R.P.F., in areas of economic analyses.

The study was funded under the Canada-New Brunswick Flood Damage Reduction Program, in turn funded jointly by the Inland Waters Directorate, Conservation and Protection Service, Environment Canada, and the New Brunswick Department of Municipal Affairs and Environment.

During the work, the study team reported to a Walker Brook Project Committee comprised of:

Chairman: Mr. Jerry G. Lockhart, P. Eng.; Program Manager of Canada-New Brunswick Flood Damage Reduction Program, New Brunswick Department of Municipal Affairs and Environment;

Mr. John D. Keefe, P. Eng.; Head, Engineering Division Atlantic Region, Inland Waters Directorate, Conservation and Protection Service, Environment Canada;

Dr. Nabil D. Elhadi, P. Eng.; Director of Water Resources Planning Branch, New Brunswick Department of Municipal Affairs and Environment;

Mr. Brian C. Burrell, P. Eng.; Hydrotechnical Engineer, New Brunswick Department of Municipal Affairs and Environment; and

Mr. David L. Sullivan, P. Eng.; Hydraulic Engineer, New Brunswick Department of Transportation.

This Project Committee provided the reference materials that were available from Phase I of the project and its related reference works, examined procedures with respect to the technical and analytical work, monitored progress, and provided constructive cross- examination and critique of the preliminary conclusions and recommendations. A total of five meetings were held between the study team and the Project Committee in conjunction with the work.

FENCO Engineers Inc. wishes to express sincerest appreciation for the considerable support, direction, and assistance provided by all members of the Project Committee during this study. In addition,we wish to acknowledge the fullest co-operation of the offices of:

Mr. Stephen T. Dohaney, P. Eng.; City Engineer, City of Campbellton;
Mr. Gilles Guérette; Director of Restigouche District Planning Commission;
Mr. Dave J. Johnstone, P. Eng.; District Engineer, District No. 1, Campbellton, New Brunswick Department of Transportation;
Mr. Herbert C. Page, P. Eng.; Director of Highway Design Branch, New Brunswick Department of Transportation;
Mr. Garth A. Rushton, P. Eng.; Director of Structures Branch, New Brunswick Department of Transportation; and
The Water Survey of Canada, Inland Waters Directorate, Conservation and Protection Service, Environment Canada.

2.2 STUDY OBJECTIVES

The purpose of the study was to identify, compare, and evaluate the engineering and economic feasibility of alternative structural and non-structural flood damage reduction measures for 1:2, 1:20, 1:50 and 1:100 year return period flooding conditions of the Walker Brook Flood Plain.

As stated in the Terms of Reference, the objectives of this study were:

1. to re-calibrate and verify the hydrologic model used in the Phase I study for this project, and to identify the magnitude of the 1:2, 1:20, 1:50 and 1:100 year return period design flows;
2. to simulate the water surface profiles associated with each return period and with each scheme, including the status quo;
3. to identify feasible structural and non-structural flood damage reduction measures or schemes;
4. to produce preliminary cost estimates of the various measures or schemes;
5. to determine average annual flood damages in the Study Area; and
6. to determine benefit-cost ratios for the various flood damage reduction measures and schemes.

2.3 METHODOLOGY

The methodology commenced with the typical "obtain all existing background data", which, in this instance, was to include the key element of the Phase I study for this project, the final HEC-2 backwater model (i.e. the as-revised 1985 version). By March 15 it was concluded that the model would not be available for this phase of the project, so it became necessary to create another one from the available data from the original study.

An initial field reconnaissance and survey was conducted to update the 1:1 200 scale Flood Risk Maps for the more highly developed, lower reaches of the flood plain. This involved tying in additional development, and in some cases deleting ones, with respect to the original base mapping, which was based on aerial photography of 1976.

Meetings were held in Campbellton with District No. 1 Transportation Engineer Dave Johnstone, P. Eng., the Director of the Restigouche District Planning Commission, Mr. Gilles Guerette, and City of Campbellton Engineer Stephen Dohaney, P. Eng., all with a view to establishing a better appreciation of flooding conditions, related damages, and planning considerations involving the Walker Brook flood plain.

In addition, meetings were held in Fredericton with senior Department of Transportation planning and design personnel in order to establish the current status of, and relevant details for, several Department projects that have a bearing on the study area.

Essentially, the original methodology that was established at the time of the engineering proposal preparation was followed throughout the execution of the assignment, only the timing suffered.

2.4 REPORT FORMAT

Due to the nature of this study and the analyses, the complete report is presented in three volumes:

- a summary report;

- this report volume, which contains the analysis of each of the subject areas of hydrology, hydraulics, flood damages, flood damage reduction options, engineering and environmental considerations, flood damage reduction costs, benefit-cost analyses, and the attendant summary conclusions and recommendations; and

- a separate Technical Appendices volume, which contains related analytical data and reference material supplementary to the study; i.e. all other material pertaining to, but not necessary for, a reading of this report volume.

3.0 HYDROLOGIC ANALYSES

3.1 GENERAL

The Water Survey of Canada, Inland Waters Directorate, Environment Canada established a gauge on Walker Brook in late 1981 (station no. 01BJ009) in response to a request from officials involved in implementing the Canada-New Brunswick Flood Damage Reduction Program. It has been operating continuously since that time, providing six years of useful hydrometric data about the study area. However, the record length is too short to be used on its own for reliably estimating peak discharges during extreme floods. As a result, several techniques were employed to augment the available data. These included:

- Statistical Analyses
  - Regional Flood Frequency Analysis (Acres, 1977)
  - Flood Frequency Analyses New Brunswick (Env. Can. and NB Dept. of Municipal Affairs and Environment, 1987)
  - Data transfer from an adjacent watershed
- Deterministic Analysis
  - Hydrologic modelling, employing an Instantaneous Hydrograph approach - HYMO (US Dept. Agriculture, 1973)

The application of each approach for estimating peak discharges for Walker Brook is discussed below.

3.2 STATISTICAL ANALYSES

3.2.1 Regional Flood Frequency Analysis (Acres, 1977)

The purpose of this investigation was to determine relationships for estimating the mean annual, 1:20 and 1:100 year flood peaks at potentially flood-prone areas. It was determined for Northern New Brunswick that peak instantaneous flows for these return periods could be calculated using the equations:

$$QP_2 = 0.586A^{0.848}\,CLS^{3.49}\,FDTS^{-0.552} \quad (+36\%, -27\%)$$
$$QP_{20} = 1.12A^{0.861}\,CLS^{5.47}\,FDTS^{-0.92} \quad (+48\%, -32\%)$$
$$QP_{100} = 1.48A^{0.865}\,CLS^{6.42}\,FDTS^{-1.08} \quad (+60\%, -37\%)$$

The parameters in these equations are:

QP = peak instantaneous flow for either the 2, 20 or 100 year return period (m^3/s),
A = drainage area (km^2),
CLS = measure of flow control by lakes and swamps, (A-AS)/A,
AS = area of lakes and swamps (km^2),
FDTS = proportion of annual maximum flood peaks occurring in March to May at the drainage basin centroid, and
(+,-) = 95% confidence limits.

Details about the development of this approach and its limitations are described elsewhere (Acres, 1977). Noteworthy, however, is that the approach is recommended for areas larger than 25 km^2 and that the slope of Walker Brook is slightly steeper than the recommended range associated with development of the above equations. Despite these limitations, this Regional Analysis was applied to portions of Walker Brook with results given in Table 3-1. Values have been estimated for the 1:50 year flows, and flows for the drainage area above Val D'Amour (too small for certain application of the Acres approach) have been noted as approximate.

3.2.2 Regional Flood Frequency Analyses (Environment Canada and New Brunswick Dept. of Municipal Affairs and Environment, 1987)

This recent regional approach (which we will subsequently call the

TABLE 3-1

WALKER BROOK - CAMPBELLTON

REGIONAL ANALYSES FOR

MAXIMUM INSTANTANEOUS FLOWS

ACRES (1977)

LOCATION AND AREA (km^2)	RETURN PERIOD (YEARS) AND FLOW (m^3/s)			
	1:2	1:20	1:50[1]	1:100
. Walker Bk near mouth (67.4)	20.1	39.1	46.3	51.8
. Walker Bk at WSC gauge (60.8) near Sugarloaf Mt.	17.8	34.4	40.8	45.4
. Walker Bk u/s of Sugarloaf (45.0)	14.3	28.6	34.2	38.2
. Walker Bk near Val D'Amour (14.0)*	5.5	10.9	13.0	14.5

CNB (1987)

LOCATION	RETURN PERIOD (YEARS) AND FLOW (m^3/s)[2]			
	1.2	1.20	1:50	1:100
. Walker Bk near mouth (67.4)	16.8 (23)	28.0 (46)	31.6 (56)	34.2 (65)
. Walker Bk at WSC gauge (60.8) near Sugarloaf Mt.	15.1 (21)	25.2 (42)	28.5 (51)	30.8 (56)
. Walker Bk u/s of Sugarloaf (45.0)	11.2 (16)	18.6 (32)	21.0 (39)	22.6 (44)
. Walker Bk near Val D'Amour (14.0)*	3.5 (5)	5.7 (11)	6.4 (14)	6.8 (16)

* values considered approximate because drainage area is much less than the range of independent variables used in developing regression equations

1 values estimated by graphical techniques

2 values from Northwestern regression equation; with values in brackets developed from the Province-wide equation

"CNB (1987)" approach) expanded on the earlier Regional Analysis. An additional ten years of measurements were available from new and ongoing gauging stations, adding significantly to the data base required for such analyses.

Province-wide and Regional regression equations were developed in these analyses for homogeneous hydrologic areas. Walker Brook and neighbouring watersheds such as the Eel River are located in the Northwestern Region. The period of record 1970-85 was selected to provide a uniform basis for comparison, because of data availability and the results of statistical analyses of long-term records.

The regression equations for estimating flood flows take the form:

$$\log_{10} QD_T = k + a\log_{10}DA + b\log_{10} MAP$$

where;

QD_T = annual maximum daily mean flow (m^3/s) with a recurrence interval of T years,
k,a,b = regression constant and coefficients,
DA = drainage area (km^2), and
MAP = mean annual precipitation (+1150mm over Walker Brook).

Significant to the projection of peak flows for Walker Brook is that the equations for the Northwestern Region were not developed for application at drainage areas smaller than 199 km^2, - the lower range of the drainage area variable used to develop the equation. However, this constraint is overcome in the Province-wide equation (for drainage areas larger than 27 km^2). Counterbalancing this is that this latter equation may give higher estimates of flow (CNB, 1987) because it must account for greater variability of stream characteristics than exist in the Northwestern Region.

Despite these considerations (similar to those in the Acres, 1977 approach), flood flow estimates were prepared from the CNB (1987) approaches and are presented for reference in the bottom portion of Table 3-1. Two estimates are given for each location and return period. The first is derived from the Northwestern equation, and those in brackets are from the Province-wide equation.

The CNB (1987) approach also presents regression equations to estimate maximum instantaneous flows from daily maxima, and the results of such study for Eel River near Eel River Crossing (Q_{inst}/Q_{daily} ratio = 1.29). Estimates of instantaneous flows, which are given in Table 3-1, reflect use of this ratio.

In summary, three regional flood flow estimates are provided in Table 3-1. Regional flow estimates in the upper portion of Table 3-1 and in brackets in the lower table are believed herein to be the more reliable estimates. The Northwestern approach, which is designed for estimating flows in much larger areas, appears to underestimate peak flows for the relatively small Walker Brook area.

Preliminary confirmation about this perception of the Northwestern approach for Walker Brook, and an indication of the variability in flood flow estimates derived from Regional analyses alone, are given below. The table compares the 1:2 year flow from an adjacent, similar watershed (Eel River, station 01BJ004) with the same flow estimated from regional analyses:

.	1:2 year flow estimate from 15 years of measured data (Eel River):	25.7 m^3/s
.	CNB Province-wide estimate:	22.4 m^3/s
.	CNB Northwestern estimate:	17.0 m^3/s
.	Acres regional estimate:	20.5 m^3/s

All of the regional estimates appear to give lower values than observed data from this basin, but the Northwestern estimate is substantially lower than the measured data. Equally important is that the other two approaches also express a variation from the observed data, - a variation which can be significantly narrowed by the following analysis.

3.2.3 Data Transfer - Eel River

The Water Survey of Canada, Inland Waters Directorate, Conservation and Protection Service, Environment Canada, has recent flow data from two locations on this nearby river:

- Eel River near Eel River Crossing (88.6 km^2)
- Eel River near Dundee (43.2 km^2)

The gauge near Eel River Crossing (01BJ004) has 16 years of peak flow data, and monitors a watershed with similar meteorological, topographic, and land use characteristics to Walker Brook. As its record is a more complete source of local, long-term data, it is an appropriate location from which to transfer peak flow estimates to Walker Brook. Significant is that there are three years of coincident stream flow monitoring of the Eel River and Walker Brook (1981-1983).

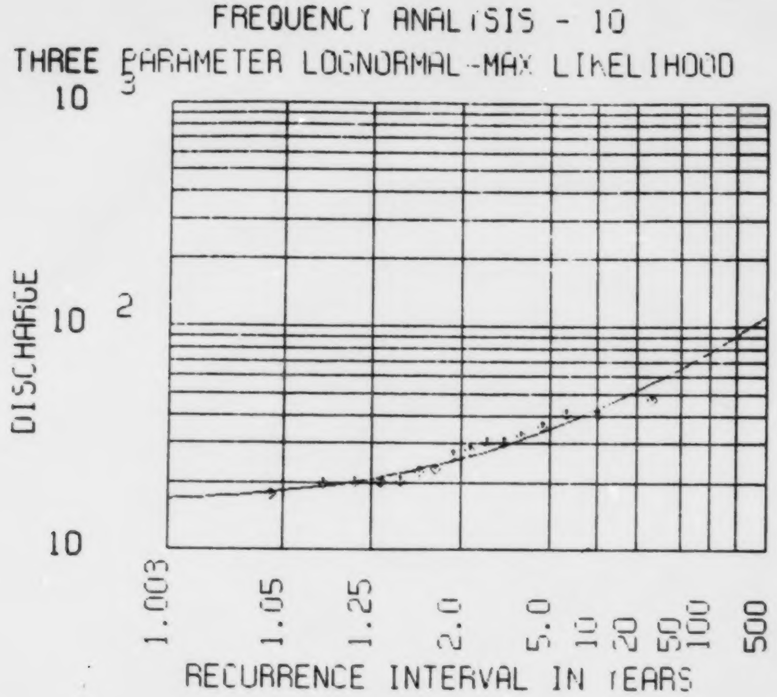
It was concluded in earlier study (SMCC, 1983) that data from Eel River near Eel River Crossing was independent and free from trend and persistance. Herein, it was further confirmed by Mann-Whitney testing for homogeneity on the nearby Upsalquitch watershed (see Technical Appendix A-3) that there is no statistical difference between the 14 years of sampled data (1969-1983) and the long term (58 years). In all, frequency analyses results using the available Eel River data base are representative of long-term flow statistics at this location.

Frequency analyses were undertaken at Eel River using the computer program CFA88 (Environment Canada, 1985), which provides flood flow estimates for a number of return periods using several different frequency distributions. The detailed results of the analyses are provided in the Technical Appendix A-1, and are summarized in Figure 3-1 and the upper portion of Table 3-2.

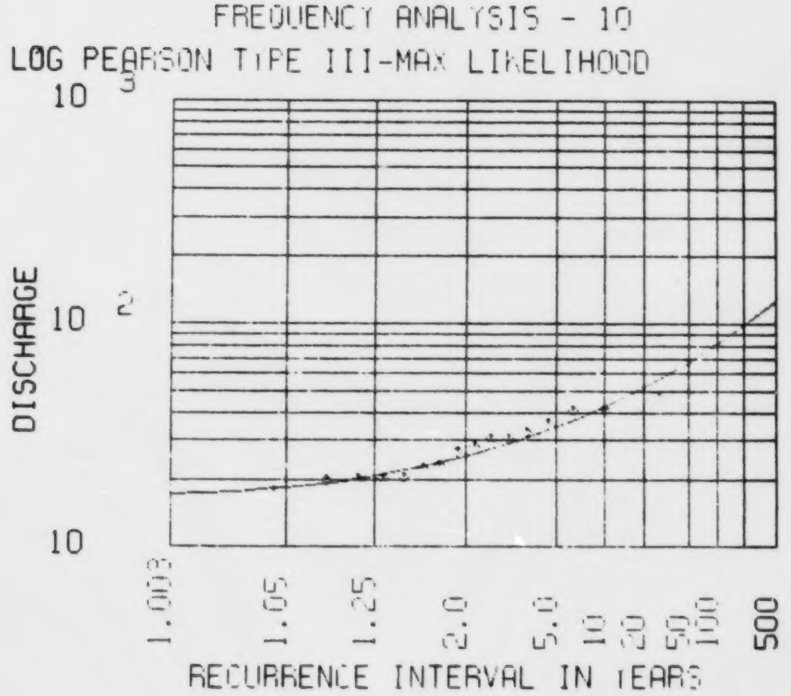
The results of the appended statistical analysis show that the Gumbel and Lognormal distributions are not good fits to the data. The Three Parameter Log Normal (3PLN) distribution and Log Pearson Type 3 (LP3) distributions have comparable standard errors and appear to give the best fits, except that the coefficient of skewness for the 3PLN distribution (-0.294) differs significantly from the theoretical value (0.0). It is also considerably outside the mean of values for the province (-0.04) and for the Atlantic Provinces in general (-0.02 to -0.08). The Wakeby Distribution is best suited for analysis of long term stations and was not considered valid for this site.

The plots of the 3PLN and LP3 distributions (Figure 3-1) emphasize the skew of the theoretical best-fit distributions and the likelihood that both would overestimate flood flows. The best fit by visual inspection to the upper peak flows is the 3PLN Distribution, although it is evident that this conclusion about "fit", based on the available 16 years of data, is a questionable one.

In order to provide additional information to support a firm conclusion about the flood frequency distribution for Eel River, the flow record near Eel River Crossing was extended by drawing on upstream records near Dundee and the findings of the CNB (1987) study. As noted earlier, the latter study projects flood flows for areas larger than 199 km^2 in the Northwestern Region (including Walker Brook) using a regression constant (k) and the logarithm of drainage area multiplied by a regression coefficient (a). A most valuable finding for the Northwestern Region is that the regression coefficient (a) is almost a constant value (1.0) for all return periods (e.g. 0.995 for the 1:2 year flood and 1.023 for the 100 year flood). Hence, for practical purposes of this study, the flood peak from a gauged site can be transposed to an ungauged site



FREQUENCY ANALYSIS - 10
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD



FREQUENCY ANALYSIS - 10
LOG PEARSON TYPE III-MAX LIKELIHOOD

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Frequency Analyses-10

FENCO **FIG. 3-1**

TABLE 3-2

WALKER BROOK
FREQUENCY ANALYSES - EEL RIVER NEAR EEL RIVER CROSSING

		RETURN PERIOD (YEARS) AND FLOW (m^3/s)			
Comment	Distrib'n	1:2	1:20	1:50	1:100
. daily peaks for 16 years observed data	3PLN	25.7	52.0	65.6	77.5
	LP3	25.5	52.0	67.3	81.5
. daily peaks for observed and transferred data	3PLN*	25.1	44.6	51.0	55.8
	LP3	25.4	43.3	48.6	52.2

* selected distribution

simply by the ratio of the drainage areas. A modest improvement would include the coefficient (a) as an exponent to the drainage area ratio.

Table 3-3 summarizes the available data from recent monitoring on the Eel River and the estimates of flow at Eel River near Eel River Crossing derived from data near Dundee. This data transposition provides four additional peak flow estimates for the WSC site near Eel River Crossing.

The four 1984 to 1987 peak daily flows derived by this transfer from the Dundee station were next added to the observed data at Eel River Crossing and the frequency analysis of flood flows was repeated. Results are presented in the bottom lines of Table 3-2 and in the Technical Appendix A-2.

The frequency analysis including the transferred data from the Dundee gauge greatly clarifies the projection of peak daily flows near Eel River Crossing. Again the Gumbel and Lognormal Distributions provide a poor fit to the statistics (Technical Appendix A-1). The 3PLN and LP3 Distributions, shown in Figure 3-2, provide an excellent visual fit and acceptable statistics. The coefficients of skewness and kurtosis for the 3PLN Distribution (-0.053 and 3.208) are close to theoretical values and the values which are typical for the Province.

Unquestionably, the extension of the data base near Eel River Crossing gives confidence to the selection of a peak flow distribution for the area. The 3PLN Distribution is selected as it has a slightly lower standard error and is a better fit to the peak flood flows. The estimated maximum daily flows at the gauge near Eel River Crossing are noted in Table 3-2.

3.2.3.1 Maximum Instantaneous Discharge. The daily mean maximum flows from the frequency analysis were converted to instantaneous flow estimates by evaluating the ratios of recorded instantaneous

TABLE 3-3

WALKER BROOK
PEAK FLOW DATA SUMMARY

	PEAK DAILY FLOW (m^3/s) AT:			
Date (yr,mo,day)	Walker Brook ($60.8\ km^2$)	Eel River (Crossing) ($88.6\ km^2$)	Eel River (Dundee) ($43.2\ km^2$)	Regression Coef. (a)
81 Apr 10	$\underline{8.94}$	15.6	-	
81 Apr 30	7.90	14.3	-	
82 Apr 29	8.96	16.3	-	
82 May 09	$\underline{12.40}$	$\underline{17.8}$	-	
82 Jul 30	2.78	4.3	-	
83 May 01	$\underline{13.60}$	$\underline{20.1}$	-	
83 Jun 01	3.87	5.6	-	
84 May 01	$\underline{15.00}$	$\underline{27.5}$*	$\underline{13.50}$	(.990)
84 May 09	14.00	25.8*	12.70	(.985)
85 Apr 31	$\underline{5.32}$	$\underline{10.9}$*	$\underline{5.55}$	(.940)
85 May 19	4.59	7.7*	3.90	(.940)
86 Apr 28	$\underline{11.50}$	$\underline{18.1}$*	$\underline{9.05}$	(.965)
86 May 22	6.56	9.6*	4.87	(.940)
87 Apr 19	$\underline{7.68}$	$\underline{14.1}$*	$\underline{7.11}$	(.950)

- no monitored data

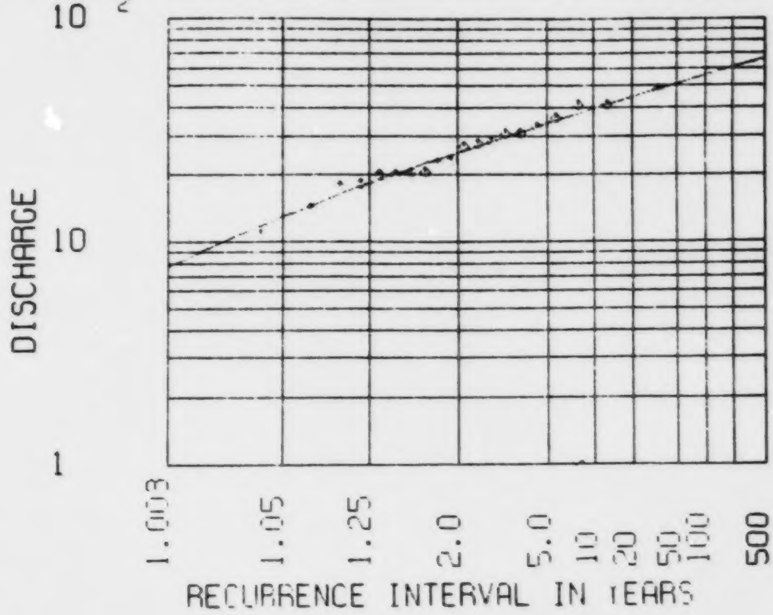
* estimated from Eel River near Dundee (01BJ012)

(-) estimated regression coefficient

___ maximum annual peak daily flow

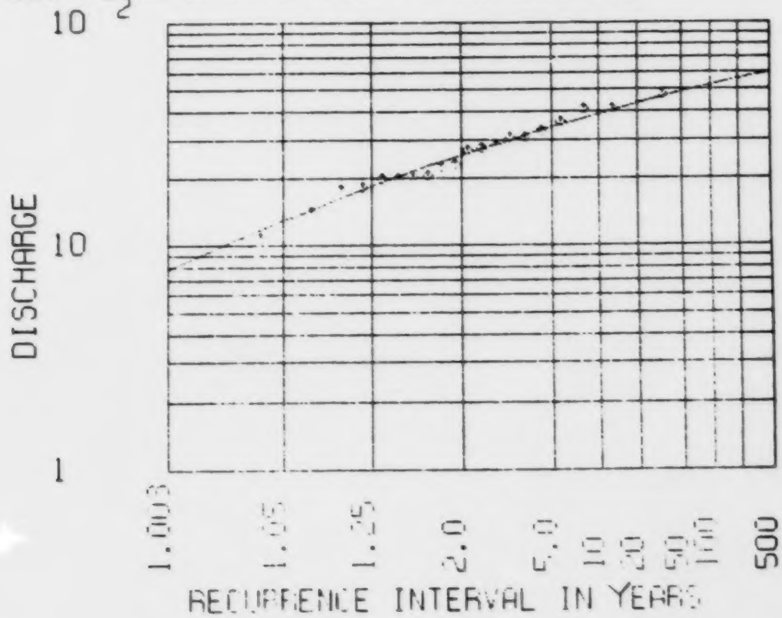
FREQUENCY ANALYSIS - 11

THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD



FREQUENCY ANALYSIS - 11

LOG PEARSON TYPE III-MAX LIKELIHOOD



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Frequency Analyses-11

FENCO **FIG. 3-2**

to daily values. (Listed in Table 3-4 and plotted as a scatter diagram in Figure 3-3). The figure suggests a trend to higher ratios with higher flows and, hence, the maximum observed ratio (1.29) was selected for determining conservative peak instantaneous flood flows near Eel River Crossing. This ratio is also similar to values determined for Walker Brook where the data is limited.

Application of this ratio to Eel River Crossing gives the following instantaneous peak flows for selected return periods.

1:2 year	1:20 year	1:50 year	1:100 year
32.4 m^3/s	57.5 m^3/s	65.8 m^3/s	72.0 m^3/s

These values are almost identical to the 1:20 year (56.5 m^3/s) and 1:100 year (72.1 m^3/s) values determined by SMCC in their independent, detailed analysis reported in 1983.

3.2.3.2 Flow Transfer to Walker Brook. There are several techniques which may be used to transfer flow data from Eel River to sites (like Walker Brook) which lack a sufficient data base for generating site-specific flood flow estimates. These can employ ratios of peak from regional analysis (e.g. SMCC, 1983), but herein we initially drew on relationships between the coincident record of flows at both stations.

Flows in the period from 1981 to date were monitored on Walker Brook and the Eel River. The WSC station near Eel River Crossing spans from 1981 to 1983 when the station was moved to near Dundee (1984-present). Regression analyses of flows listed in Table 3-3 were first conducted for the 1981-83 period and then for the entire period (using the transferred data from Dundee (Table 3-3). Both analyses indicate that flows can be reliably transferred from the Eel River watershed using the general formula:

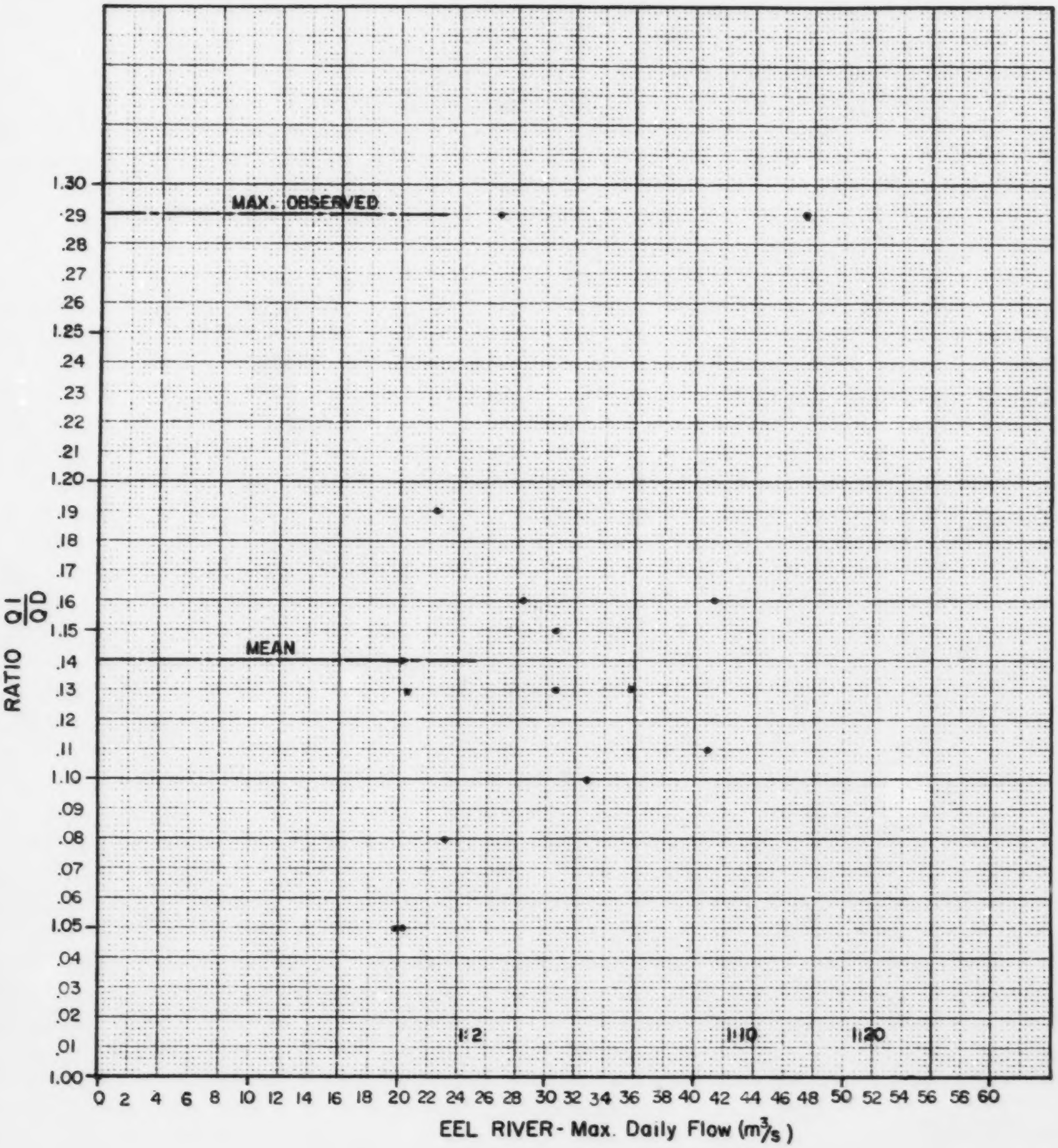
$$Qw = C(Qe)^N$$

TABLE 3-4

WALKER BROOK

DEVELOPMENT OF MAXIMUM INSTANTANEOUS FLOWS

Year	Maximum Instantaneous	Maximum Daily	Ratio Qi/Qd
	Eel River Discharge (m^3/s)		
1968	34.5	30.6	1.13
1969	40.5	35.7	1.13
1970	60.9	47.3	1.29
1971	34.5	26.7	1.29
1972	25.0	23.1	1.08
1973	44.7	40.8	1.10
1974	32.6	28.2	1.16
1975	36.2	32.8	1.10
1976	21.3	20.3	1.05
1977	23.3	20.6	1.13
1978	26.6	22.4	1.19
1979	47.6	41.1	1.16
1980	35.1	30.6	1.15
1981	20.9	20.0	1.05
1982	-	17.8	-
1983	22.9	20.1	1.14
		mean:	1.14
		maximum:	1.29
	Walker Brook Discharge (m^3/s)		
1981	10.70	8.94	1.20
1982	15.20	12.40	1.23
1983	16.60	13.60	1.22
1984	-	15.00	-
1985	5.80	5.32	1.09
1986	13.90	11.50	1.21
1987	8.09	7.68	1.05
		mean:	1.17
		maximum:	1.23



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Instantaneous to Daily Flow Ratios, Eel River

FENCO

FIG. 3-3

where Qw = Walker Brook flow estimate (m^3/s),
Qe = Eel River flow estimates (m^3/s),
C = regression constant, and
N = regression exponent.

Values for the parameters C and N vary with the period of record employed in the analysis and are listed below with the coefficient of correlation.
Additional plots and statistics for each regression are contained in Technical Appendix A-2.

Period	C	N	Coefficient
1981-83	0.7111	0.9472	0.9845
1981-87	0.7186	0.9288	0.9779

Application of the findings of either period gives similar flow estimates for the Walker Brook, although the equation for the initial period (1981-83) gives slightly higher flow estimates (above 7%) than the period which includes transposed data from upstream.

This correlation is based on flow comparisons during the 1980's, which provided a sequence of relatively low values of the annual flood peaks. Only one flood peak was in the order of the 1:2 year flows.

In order to determine if regressions developed for 1980's flows would be valid for the longer term (containing higher flows), two similar regression analyses were conducted with data from nearby river systems.

- Eel River Crossing data was compared to that of the Jacquet River to determine if 1981-86 regressions would be similar to 1968-86 values

- Upsalquitch River data was similarly compared to the Restigouche below Kedgwick River (1963-86).

These regression analyses, in Technical Appendix A-3, showed that use of the 1980's data is useful for flow transfer, but demonstrate that the 1980's estimates should be increased by approximately 8.2% to represent long-term relationships (Table 3-5). Hence, for transferring peak flows from Eel River to Walker Brook, the following conservative relationship was selected:

$$Qw = 0.7111\,(Qe)^{.9472} \times 1.082$$

This gives the following instantaneous flood flow estimates at the gauge on Walker Brook, and the upper 95% confidence limits, in brackets. (Also see Appendix A-5.)

1:2 year	20.8 m^3/s (23.8 m^3/s)
1:20 year	35.7 m^3/s (46.2 m^3/s)
1:50 year	40.6 m^3/s (55.5 m^3/s)
1:100 year	44.2 m^3/s (62.2 m^3/s)

Comparison with values presented earlier in Table 3-1 shows that these values are extremely close to those projected by the Acres (1977) regional analysis. They also fall within the range of anticipated values using the CNB (1987) approach.

3.3 DETERMINISTIC ANALYSIS

3.3.1 General

The maximum annual flows at the gauge on Walker Brook (1981-1986) and the nearby Eel River (1968-80) have occurred in April and May. The adjacent Upsalquitch River shows a similar predominance of spring flood peaks, but includes six annual maximum flows (in the 39 years before 1967) which were not spring events. These occurred in June and in the fall and two were in the range of the 1:10 year flood.

TABLE 3-5
WALKER BROOK - CAMPBELLTON

REGRESSION COMPARISONS

Upsalquitch from Restigouche River*			Long Term difference from Short Term
1:20 yr	1981-86	635 m^{3}s	12.1% larger
	1963-86	712 m^{3}s	
1:100 yr	1981-86	756 m^{3}s	12.6% larger
	1963-86	851 m^{3}s	
Eel Crossing from Jacquet River **			
1:20 yr	1981-86	39.7 m^{3}s	5.6% larger
	1968-86	41.9 m^{3}s	
1:100 yr	1981-86	50.7 m^{3}s	2.3% larger
	1968-86	51.9 m^{3}s	
		Range:	2.3 - 12.6%
		Average:	8.2%

* 1 300 m^3/s and 1 600 m^3/s taken to be 1:20 and 1:100 year flows

** 200 m^3/s and 250 m^3/s taken to be 1:20 and 1:100 year flows

Although it is evident that spring runoff has dominated the recent flood history in the watershed, extreme rainfall events may also cause high flows in the Campbellton area. Storms over nearby areas in August 1981 and August 1976 (Hurricane Belle) are good examples.

In order to estimate the magnitude of flood flows generated predominantly by rainfall, streamflow was simulated using the hydrologic computer model named HYMO (U.S. Dept. Agric., 1973). This model employs factors such as soil and land use to simulate surface conditions in the watershed, and the model generates a runoff hydrograph in response to meteorological inputs. This model was initially prepared by SMCC (1983) for earlier studies and the details of its preliminary development are well described in their report. Noteworthy, is that sufficient streamflow data is now available to allow calibration and verification of the model, and its subsequent use to estimate runoff from precipitation.

3.3.2 Meteorological Data

Meteorological data was assembled from the Atmospheric Environment Service (AES) of Environment Canada to:

- calibrate and verify the hydrologic model,
- simulate runoff from large rainfall events with snowmelt, and
- simulate runoff from major rainfall events (e.g. 1:100 year storm).

3.3.2.1 Calibration and Verification Data. Data for calibration and verification were selected from AES records for periods when relatively high streamflows resulted from rainfall input. There is not a continuously recording rainfall gauge in the watershed and, hence, representative rainfall data was gathered from stations at Charlo Airport, St. Alexis de Matapedia and Tidehead. The first two stations provide continuous rainfall records and daily totals are available from Tidehead.

Events selected for monitoring and verification of the HYMO model included:

- November 25, 1983
- June 25, 1981
- June 25, 1985
- June 9, 1981

3.3.2.2 Rainfall with Snowmelt. Rainfall with snowmelt was examined in detail in earlier studies by SMCC (1983). Therein, rainfall in the snowmelt period was separated from the annual rainfall series and was analyzed to give totals (for various return periods) which would be expected during snowmelt. The average snowmelt was then computed by approved techniques (e.g. MacLaren, 1979) and was summed with rainfall to give a total of melt plus rainfall.

Herein, that work has been extended to provide 12-hour totals for the range of return periods given below:

Return Period and Rainfall + Melt Total	1:2	1:20	1:50	1:100
12 hr total (mm)	22.5	32.4	37.0	40.8

It is appropriate to note that the rainfall totals projected from the earlier analysis for the melt period are significantly less than those which occur when there is typically little or no snowcover (i.e. late April to late November, reference: Potter, 1965). These annual rainfall events are discussed below.

3.3.2.3 Design Rainfall Distributions. The Canadian Climate Centre (CCC) of Environment Canada's Atmospheric Environment

Service (AES) has recently done considerable research into design rainfall distributions (e.g. Hogg 1980, 1982; Watt et al, 1986). For this study, we focussed on synoptic scale, cyclonic events of 12-hour durations, identified in earlier work (Spear, MacDonald, 1983) as being most appropriate for this watershed.

The distribution of rainfall for storms in the range of 12-hour durations (the AES Distribution) evolved from study of 1-hour and 12-hour rainfall characteristics, and examination of regional differences in these characteristics (Hogg, 1982a and 1982b). As part of this latter work, Hogg prepared data sheets which summarize the analysis of 12-hour storms at a number of selected locations in Canada. Charlo Airport (Station 8100880), which has now a 28 year record of rainfall rate (and which is about 25 km east of Campbellton), was one of the selected stations. It is also the nearest station with long-term data describing rainfall intensity in the Campbellton area.

Table 3-6 summarizes the distribution of total design rainfall for the 12- hour storm at Charlo as determined by this AES Distribution (Figure 3-4 shows the same data contained in data format in Technical Appendix A-4). This is based on the assessment of 48 major storms (12-hour totals in excess of 27.7 mm) which occurred in the period from late April to late November 1959-1979.

Return period rainfall amounts for the 28 year period to 1987 were also obtained from the AES for Charlo Airport (Env. Canada, 1988). These are also provided in Table 3-6 and are plotted in Figure 3-5. The 1:20 year value is not given by Environment Canada analyses but can be accurately estimated from the figure to be 70.2 mm. The 95% confidence limits (mm) are also provided in the table for selected return periods.

Lastly, it is considered appropriate (Hogg, p.c. 1987) to distribute the precipitation within the peak hours of the 12-hour

TABLE 3-6

WALKER BROOK - CAMPBELLTON
DESIGN RAINFALL DATA *

12-Hour Storm

Time (hours)	Percent of [1] Total Rainfall (%)
00-01	1
01-02	1
02-03	2
03-04	5
04-05	10
05-06	17
06-07	24
07-08	22
08-09	11
09-10	5
10-11	1
11-12	1
12 hr	100%

12-Hour Duration Storms

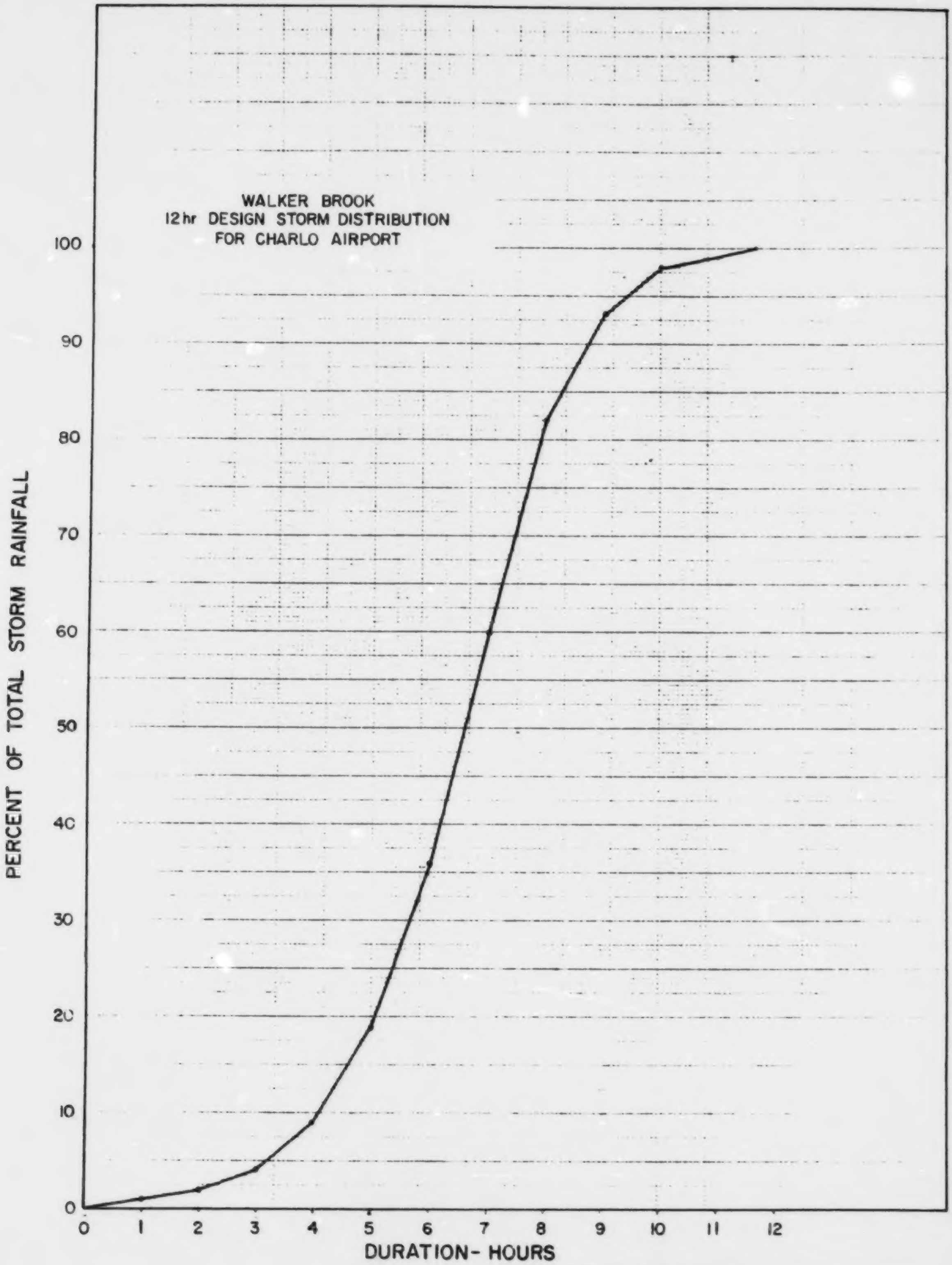
Return Period (yrs)	Rainfall [2] Amount (mm)	95% Confidence Limits (mm)
1:2	43.3	+/- 4.8
1:5	55.0	
1:10	62.7	
1:20	70.2[3]	+/- 12.6
1:25	72.5	
1:50	79.7	+/- 16.8
1:100	86.9	+/- 19.2

* at Charlo

1/ reference (Hogg, 1982b)

2/ reference (Env. Canada, 1988)

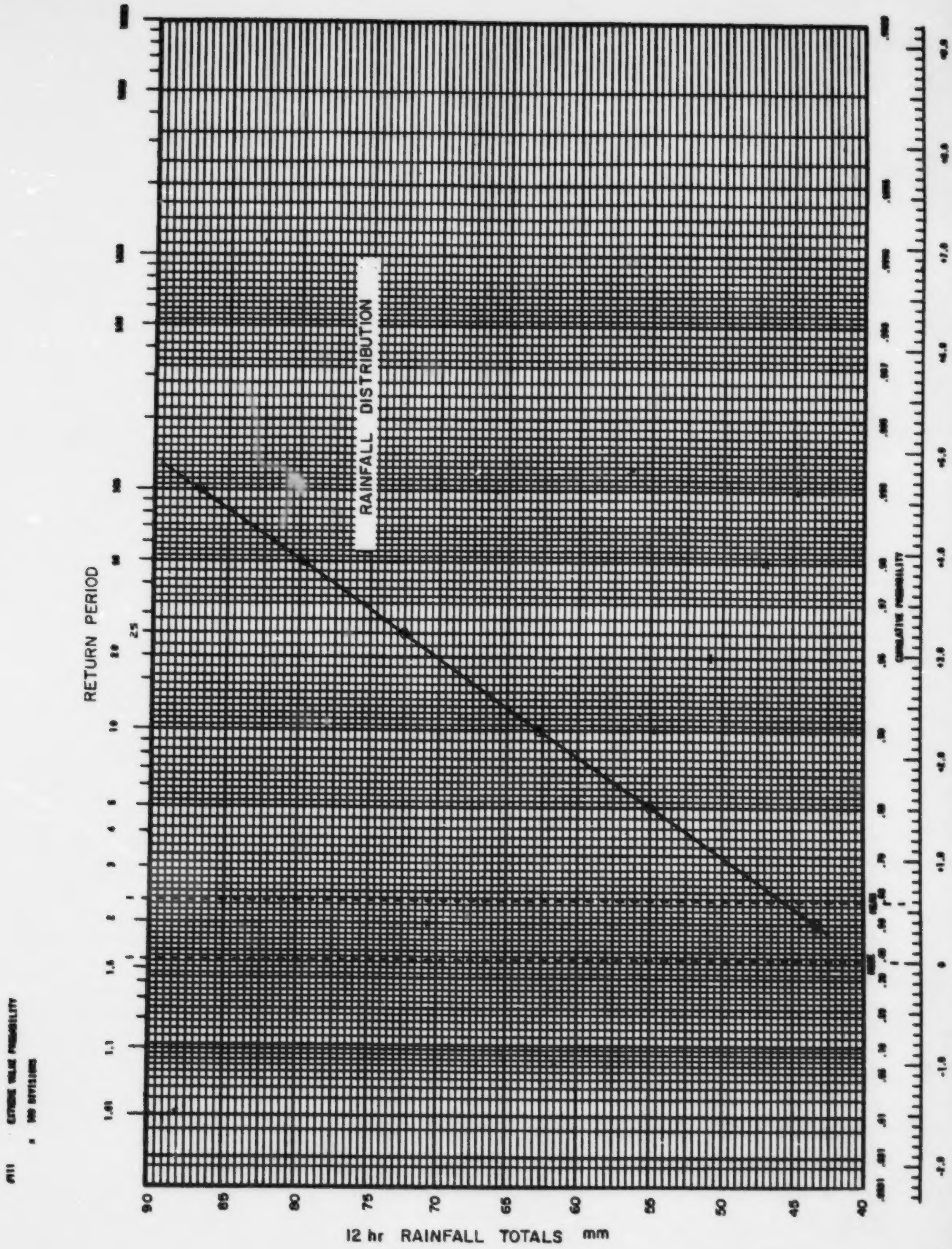
3/ interpolated from given values plotted in Figure



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Walker Brook 12-hr Design Storm Distribution

FENCO FIG. 3-4



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FLOOD PLAIN STUDY PHASE II

Charlo Airport Rainfall Distribution

FENCO **FIG. 3-5**

storm in a manner which is similar to short-term, convective storm events. This approach is described by Watt et al (1986) and it slightly modifies the 12-hour distribution to provide a peak within the seventh hour.

In effect, this modification places 60% of the rainfall of the peak hour within a 20 minute interval in that hour.

Table 3-7 summarizes the hourly rainfall amounts for each of the 1:2, 1:20, 1:50 and 1:100 year return periods. As noted above, these distributed amounts are valid for storms from late April to late November.

3.3.3 HYMO Model - Calibration and Verification

3.3.3.1 Model Set-up. The HYMO model is initially established to realistically represent the soils, land use, lakes and reservoirs and connecting channels within the watershed. This was completed previously (SMCC, 1983) and most of that work was incorporated in this analysis. Figure 3-6 shows the sub-catchments and the connectivity of streamflow between these areas and the main stream.

3.3.3.2 Model Calibration. Two events were selected to calibrate the model parameters, so that simulated rainfall runoff would closely correspond to the observed data. The calibration periods were:

- November 25, 1983
- June 25, 1981

During calibration, it became evident that the absence of continuous rainfall records within the Walker Brook basin and incomplete records at nearby gauges presented a challenge for calibration. Rainfall records from the nearby gauges gave good

TABLE 3-7

WALKER BROOK - CAMPBELLTON
DESIGN RAINFALL DISTRIBUTION

RETURN PERIOD AND RAINFALL TOTALS PER HOUR

Hour	%*	1:2 year (mm)	1:20 year (mm)	1:50 year (mm)	1:100 year (mm)
1	1	.433	.702	.797	.869
2	1	.433	.702	.797	.869
3	2	.866	1.404	1.594	1.738
4	5	2.165	3.510	3.985	4.345
5	10	4.330	7.020	7.970	8.690
6	17	7.361	11.934	13.549	14.773
7	24	10.392	16.848	19.128	20.856
8	22	9.526	15.444	17.534	19.118
9	11	4.763	7.722	8.767	9.559
10	5	2.165	3.510	3.985	4.345
11	1	.433	.702	.797	.869
12	1	.433	.702	.797	.869
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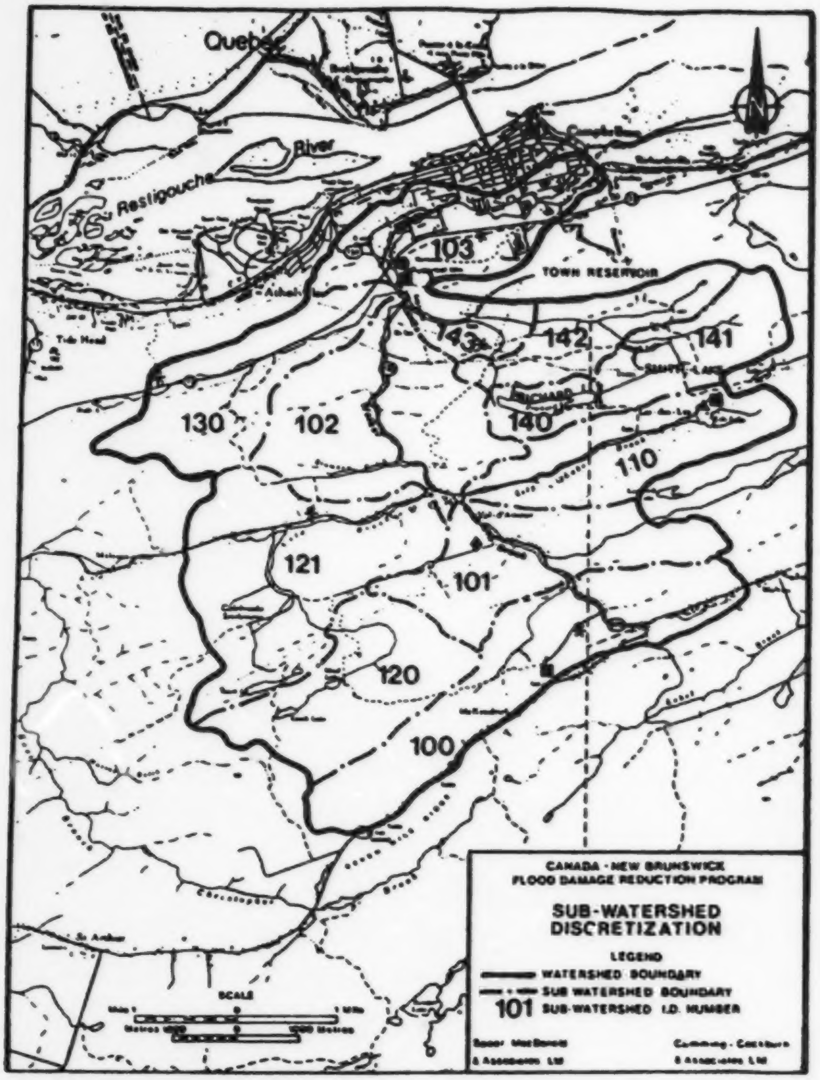
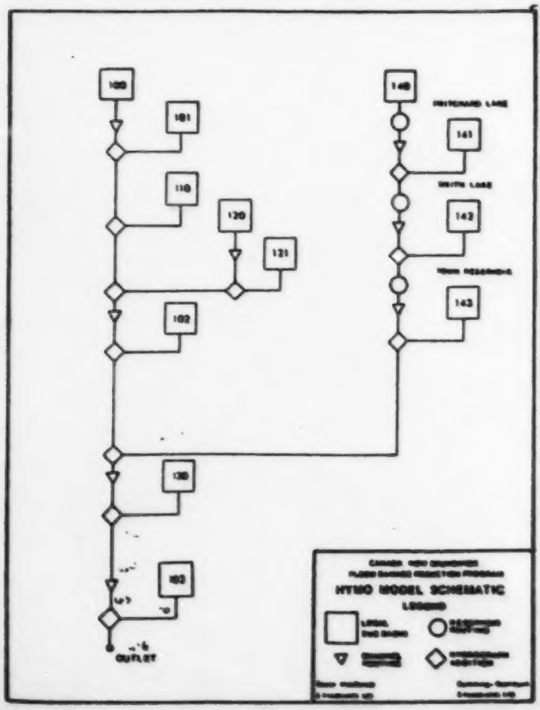
	TOTAL:	43.3	70.2	79.7	86.9

* AES hourly distribution for 12-hr storm at Charlo Airport





WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Sub-Watershed Discretization
and HYMO Model Schematic

FENCO FIG. 3-6

estimates of total rainfall and its distribution about the peak hours of a storm but limited (sometimes conflicting) information about the time when each storm was centered over Walker Brook.

As a result, it was possible to accurately simulate peak flows from storm rainfall in the area, the principal purpose of the study. It was not always possible to correctly simulate the time of the peak flow, simply because of uncertain knowledge about the time of travel of storm fronts from Charlo or St. Alexis de Matapedia to Walker Brook. Consequently, the calibration process involved a series of adjustments to model coefficients until:

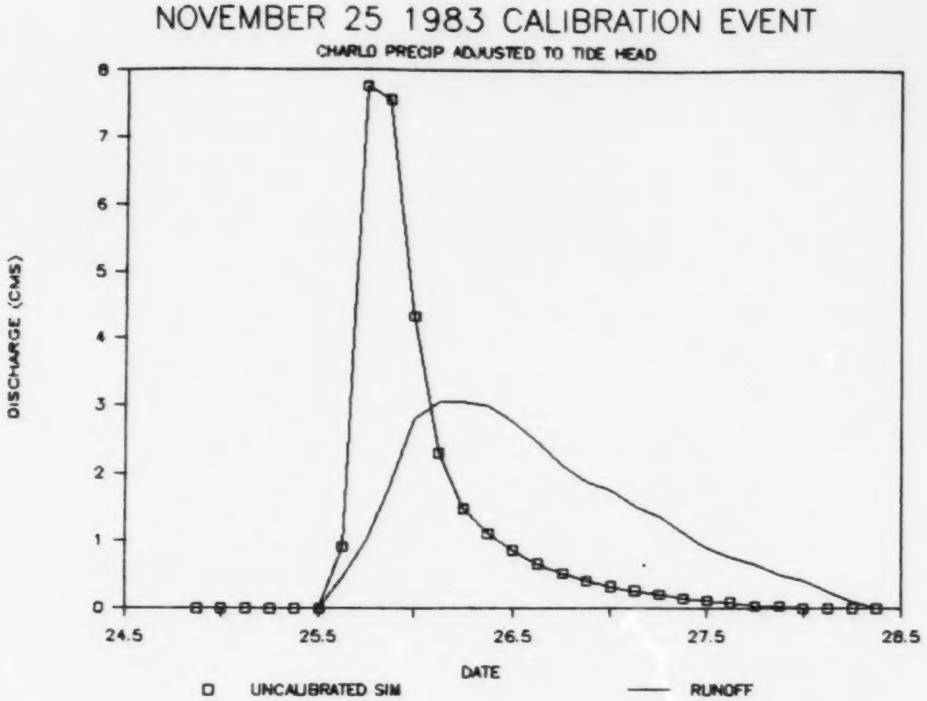
- the peak flows from the rainfall events were in close agreement with the observed peaks (the primary objective), and

- the shapes of the runoff hydrographs were reasonably well characterized.

Figure 3-7 shows the initial, uncalibrated results at the top of the Figure (SMCC, 1983) and the good fit resulting from calibration at the bottom of the Figure (event of November 1983). It should be noted that this and other figures of storm runoff do not include the base flow which is present throughout the event.

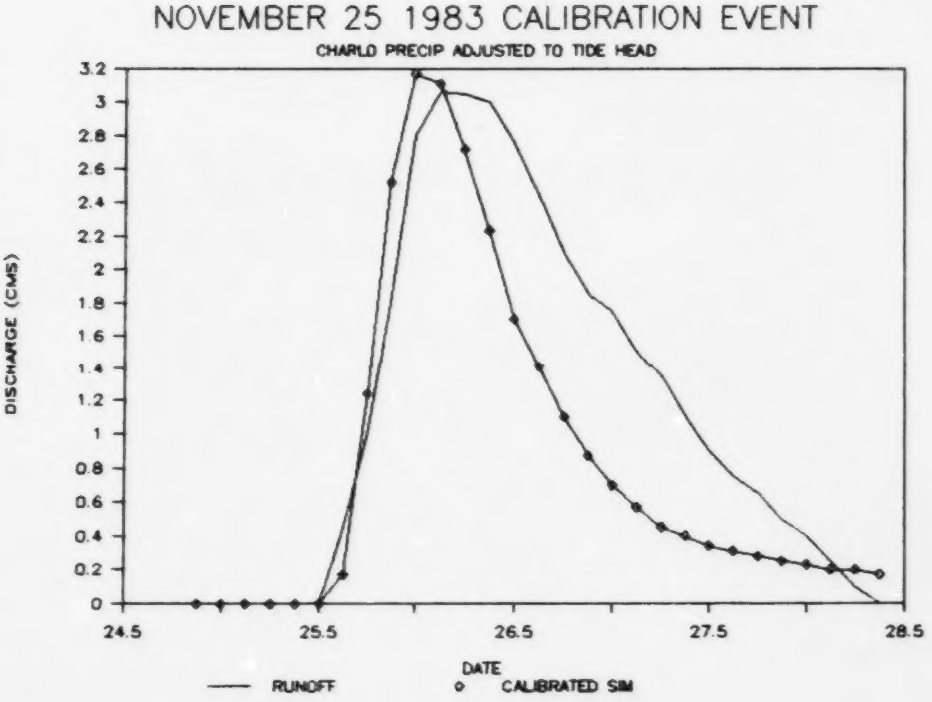
Figure 3-8 shows the event of June 25, 1981, and the results of employing rainfall from nearby stations on the timing of flows at Walker Brook. The St. Alexis de Matapedia rainfall data gives a runoff configuration which is clearly similar to the observed flow, - yet the timing is inaccurate. In this case, it is likely that the rainfall front arrived earlier at Walker Brook or followed an east-west path along the Restigouche Valley. The bottom of Figure 3-8 shows the simulated results transposed in time to overlap the observed flow.

Both of the calibration cases provided acceptable results and model



NOVEMBER 25 1983 CALIBRATION EVENT

CHARLO PRECIP ADJUSTED TO TIDE HEAD



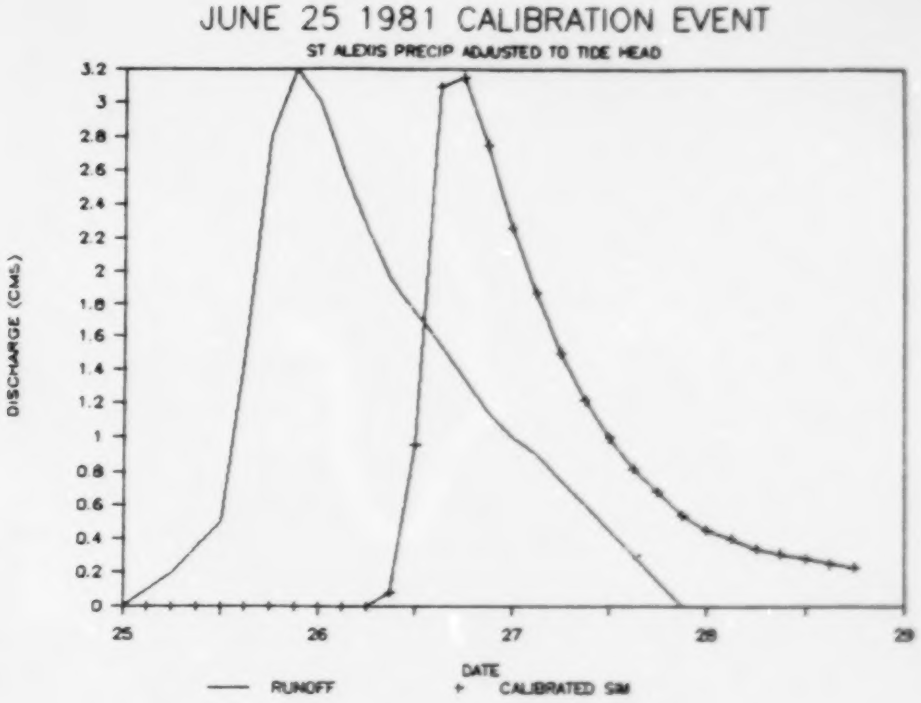
NOVEMBER 25 1983 CALIBRATION EVENT

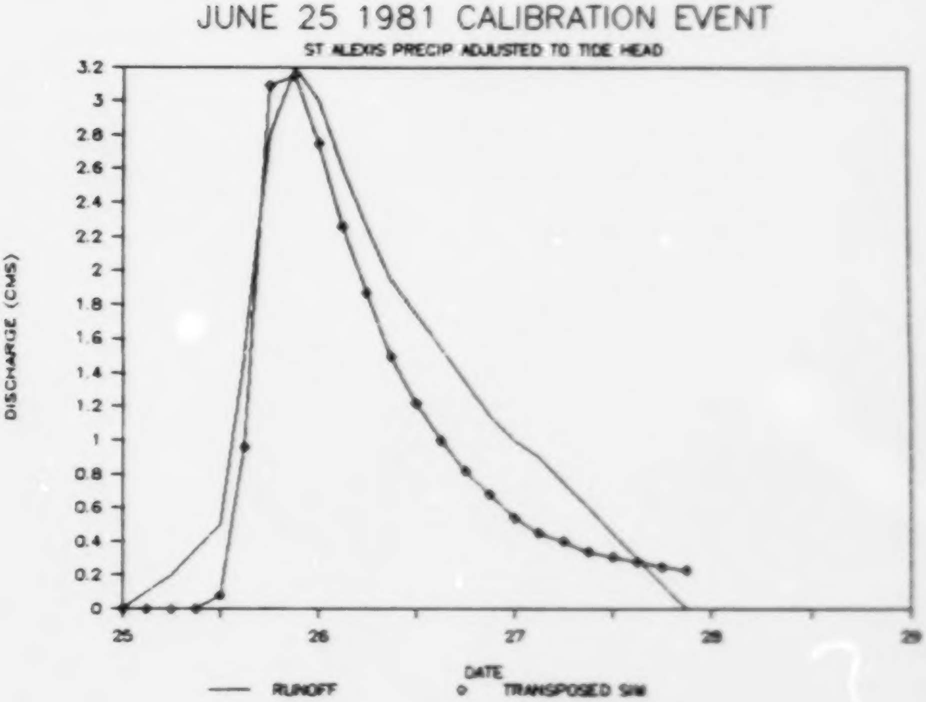
CHARLO PRECIP ADJUSTED TO TIDE HEAD

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

November 25, 1983
Calibration Event

FENCO **FIG. 3-7**





WALKER BROOK
FLOOD PLAIN STUDY PHASE II

June 25, 1981
Calibration Event

FENCO FIG. 3-8

parameters were fixed prior to verification. Table 3-8 summarizes these parameters.

3.3.3.3 Model Verification. Rainfall from the storm events of:

- June 25, 1985
- June 9, 1981

were input to the calibrated model to give a second set of modelled streamflows. The object here was to confirm that the parameters established in the calibration process (Table 3-8) were also suitable to give good estimates of peak flows for other rainfall inputs. The results were quite reasonable.

Figure 3-9 presents the June 25, 1985 simulation. The simulated peak flow is within 3% of the observed value and the shape of the hydrograph and runoff values are reasonably well represented. Again there is some transposition in time because the precipitation inputs are not local to Walker Brook.

Figure 3-10 shows the June 9, 1981 case. Here again the rainfall input derived from Charlo presents a good estimate of volumes and timing within the storm, but is not synchronous with the observed flow data. Here also it is likely that the storm over Walker Brook was similar in all respects to those at nearby AES stations, - only it occurred slightly earlier over the watershed.

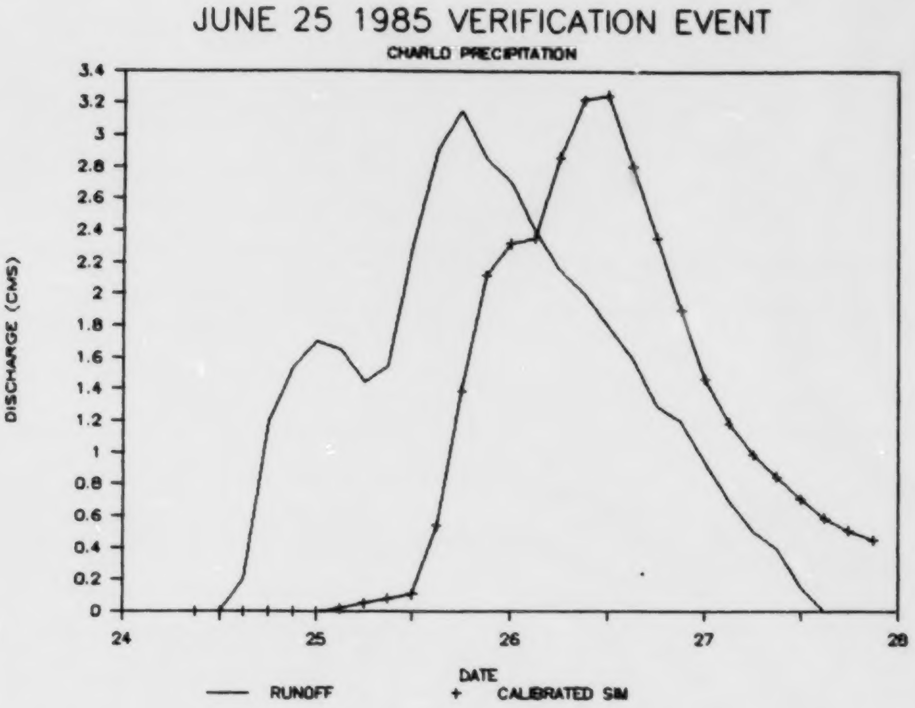
In all, it is concluded that these verifications runs (combined with the calibration cases) convincingly demonstrate that the HYMO model is suitable for simulating the peaks of rainfall runoff on Walker Brook. Inconsistencies resulting from the timing of rainfall inputs over the watershed are irksome, but not significant in view of the excellent correlation between observed and simulated peak flows. As a result, the HYMO model may be used with confidence to simulate flood flows from other storm events.

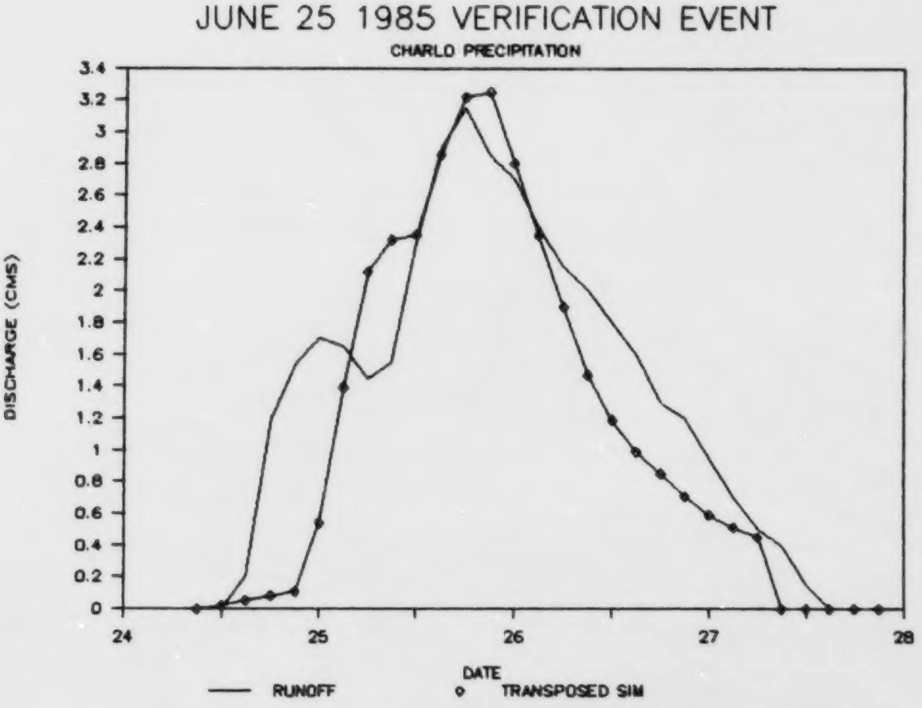
TABLE 3-8

WALKER BROOK - CAMPBELLTON

Walker Brook Basin Hydrologic Parameters - Calibrated/Verified Model

Sub-Basin	Area (km^2)	Runoff Curve Numbers AMC II	AMC III	K (hours)	Tp (hours)
100	8.73	78	90	14.40	9.68
101	5.31	77	89	11.80	8.00
110	5.62	70	85	9.92	8.56
120	5.60	78	90	10.88	8.08
121	12.44	75	88	10.72	10.48
102	7.33	77	89	15.23	9.56
130	5.39	71	86	9.6	1.96
140	1.42	93	98	24.77	8.64
141	4.12	66	82	7.87	6.40
142	4.04	78	90	7.87	5.04
143	1.94	74	88	9.89	7.92
103	5.48	86	94	7.62	7.35

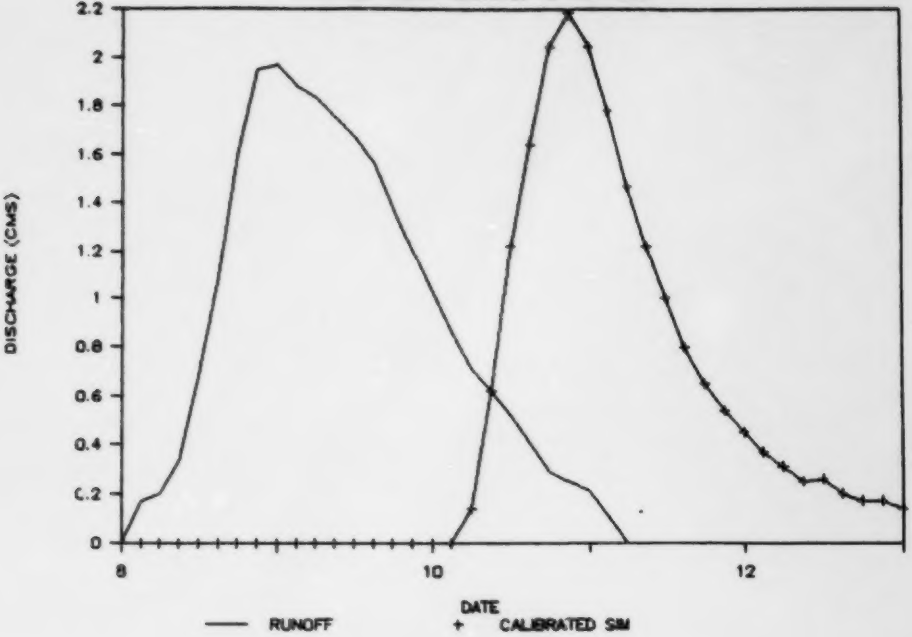




WALKER BROOK
FLOOD PLAIN STUDY PHASE II

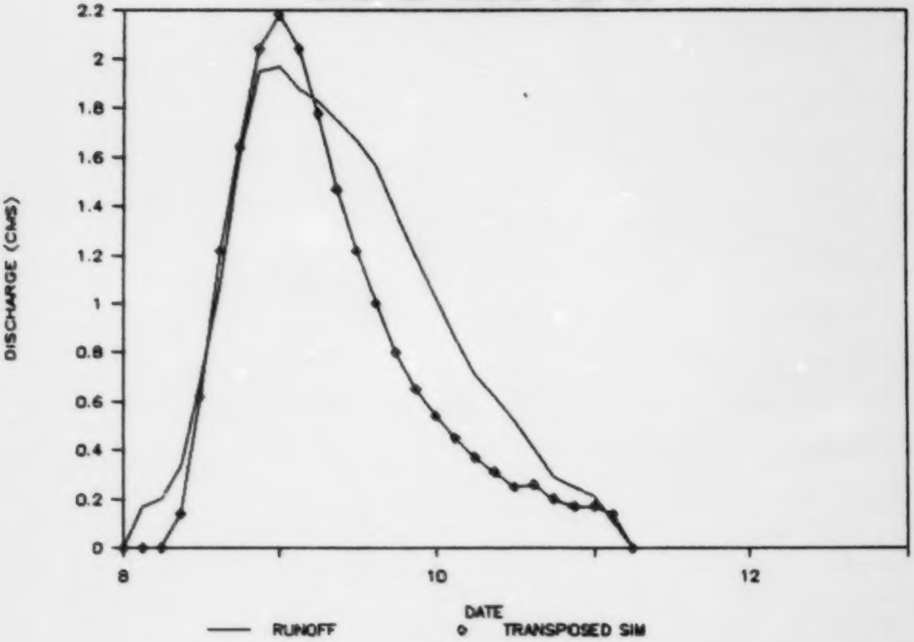
June 25, 1985
Verification Event

FENCO FIG. 3-9



JUNE 9 1981 VERIFICATION EVENT

CHARLO PRECIP ADJUSTED TO TIDE HEAD



JUNE 9 1981 VERIFICATION EVENT

CHARLO PRECIP ADJUSTED TO TIDE HEAD

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

June 9, 1981
Verification Event

FENCO **FIG. 3-10**

3.3.4 Flood Flow Simulations

Three situations may be simulated with the verified hydrologic model:

- rainfall and snowmelt events typical of the spring runoff case,
- annual rainfall events following closely on the heels of spring runoff, and
- annual rainfall events and historic events occurring in the post snowmelt period (summer and fall).

The first situation includes many possibilities ranging from minor snowmelt with light rains to significant melt with heavy rains. These combinations are too numerous and varied to attempt to simulate and are appropriately assessed in the previous statistical analyses of spring runoff.

It was noted earlier in this section that one example of melt with rainfall had previously been studied (SMCC, 1983) and found equivalent to precipitation inputs ranging from 22.5 mm to 40.8 mm over a 12 hour period (1:2 to 1:100 year). Simulations with these examples (SMCC, 1983) developed flood flows which fell within the range of anticipated spring floods but were less than those derived from the statistical analysis, Table 3-1.

3.3.4.1 Annual Event. There is little doubt that the effects of snowmelt have considerable bearing on flood flows on Walker Brook. Either the melt combines with rain or it saturates the soil surface to establish conditions which may lead to significant runoff from design rainfall events.

Alternatively, similar rainfall events may occur when soil conditions are dryer such as in the hurricane season, July to September. Overall, runoff will be higher for a given event when the antecedent moisture condition (AMC) of the soil is higher. The average soil complex numbers for Walker Brook under average antecedent soil moisture conditions (AMC-II) and wet conditions (AMC-III) were given earlier in Table 3-8.

Extensive analysis of moisture conditions was not deemed necessary herein to establish AMC conditions prior to design rainfall events. Technical Appendix A-4 identifies that major precipitation occurrences have taken place in late April and early May when snowmelt is usually just finishing, and when it is reasonable to assume a saturated soil condition in the watershed (AMC-III). In late spring and summer, it is more reasonable to consider average moisture conditions (AMC-II) to reflect the cyclical drying between rainfall events.

3.3.4.2 Design Rainfall Simulations - Spring Conditions. The design rainfalls summarized in Table 3-6 were applied through the calibrated-verified HYMO program to estimate flood peaks when the ground surface is saturated. Peak flow results at several locations are given in the upper portion of Table 3-9.

The results provide an interesting comparison with those developed by regional approaches (Table 3-1). All the simulations with saturated soil conditions (Table 3-9) are very close to the Acres (1977) results, and similar at the 1:2 year return period to the CNB (1987), Northwestern equation. The rainfall-runoff simulations also fall within the range of the Province-wide results except for the Val D'Amour area which is only approximated by regional results.

More important is that the spring design storm simulations are similar to, but less than, those estimated by flow transfer from the Eel River (given at the end of Section 3.2.3). The transferred flows project higher discharges for the lower events (e.g. 1:2 year) because of the snowmelt component (not estimated in the rainfall simulation). The higher discharges by flow transfer are

TABLE 3-9

WALKER BROOK - CAMPBELLTON

DESIGN STORM SIMULATIONS WITH DIFFERENT SOIL MOSTURE CONDITIONS

SATURATED SOIL MOISTURE CONDITIONS (AMC III)

LOCATION AND AREA (km^2)	RETURN PERIOD (YEARS) AND FLOW (m^3/s) 1:2	1:20	1:50	1:100
. Walker Bk near mouth (67.4)	15.6	34.0	41.0	46.4
. Walker Bk at WSC gauge (60.8) near Sugarloaf Mt.	13.7	30.4	36.8	41.7
. Walker Bk u/s of Sugarloaf (45.0)	10.3	22.2	26.8	30.2
. Walker Bk near Val D'Amour (14.0)	3.2	6.8	8.1	9.2

AVERAGE SOIL MOISTURE CONDITIONS (AMC II)

LOCATION AND AREA (km^2)	RETURN PERIOD AND FLOW 1:2	1:20	1:50	1:100
. Walker Bk near mouth (67.4)	5.7	17.8	23.0	27.2
. Walker Bk at WSC gauge (60.8) near Sugarloaf Mt.	4.6	15.4	20.0	23.8
. Walker Bk u/s of Sugarloaf (45.0)	3.6	11.5	14.9	17.6
. Walker Bk near Val D'Amour (14.0)	1.2	3.7	4.7	5.5

similar to the design rain simulations, reflecting the impact of high rainfall with snowmelt.

3.3.4.3 Design Rainfall Simulations - Summer Conditions. It would be anticipated that flood peaks from the same storms would be higher in the spring than the summer (when average, AMC II, soil moisture conditions would be typical). This was demonstrated in several simulations summarized in the bottom of Table 3-9. The 1:20 year runoff is about 52% of its spring counterpart and the 1:100 year runoff is approximately 58% of spring simulations. The summer flows are still significant and would cause flooding, - particularly if associated with a hurricane event.

3.3.4.4 Historical Storm Simulation - Hurricane Belle. The design storm rainfall amounts are based on events measured at Charlo from 1959-79. Not included in this set are the peaks of long duration storms such as the August 1981 event near the Quebec - New Brunswick border, Hurricane Belle, near Edmundston in August 1976; Hurricane Gladys, the 1950 storm over Port Daniel; and the 1922 event centred between Mount Carleton and Nash Creek. These and other AES Storm Analyses since 1920 suggest that there is merit in simulating such events assuming they were centred over Walker Brook.

Hurricane Belle and the 1922 and 1950 storms are three events which centred rainfall at locations within 70 to 150 km of Campbellton. All had similar 24-hr and total storm rainfall amounts (Table 3-10) but Hurricane Belle had a much higher 12 hour rainfall (about equal to 12 hr, 100 year rainfall 86.9 mm).

Hurricane Belle was selected for simulation of flood peaks along Walker Brook, and the results are presented in the bottom portion of Table 3-10. The centre column gives simulated flood peaks when the prevailing soil moisture conditions before the storm are average/typical before such storms (AMC II). The flood peak at the WSC gauge is shown to be 32% higher than the 100 year flow

TABLE 3-10

WALKER BROOK - CAMPBELLTON

HISTORICAL* STORM ANALYSIS

Storm Location	12-hr max Rainfall	24-hr max Rainfall	Total Storm Rainfall and Duration
1922 - Nash Creek	61 mm	97 mm	199 mm in 5.2 days
1950 - Pt Daniel	58	104	184 mm in 2.5 days
1976 - Edmundston (Hurricane Belle)	84	117	179 mm in 3.0 days

HURRICANE BELLE

RUNOFF ANALYSIS FOR WALKER BROOK

Flow Point on Walker Brook	Simulated Flood Peak (m^3/s) AMC II	AMC III
- at mouth (67.4 km^2)	62.3	75.4
- at WSC gauge (60.8 km^2)	56.2	68.8
- upstream of gauge (45.0 km^2)	41.0	49.9
- near Val D'Amour (14.0 km^2)	12.7	15.2

---

* After Environment Canada, Mass Curves of Rainfall - Storm Rainfall Canada

indicated by statistical analyses and 68% higher at Val D'Amour. These peaks are with the 95% confidence interval of the 100 year flood flows summarized at the end of Section 3.2.3.

The right-most column gives a similar set of results, but ones for the hurricane occurring on the watershed when it is unusually wet/saturated from previous heavy rainfall (AMC III conditions). These exceptional circumstances give flood flows which are slightly higher than the upper 95% values given at the end of Section 3.2.3.

3.4 SELECTED FLOOD FLOWS

3.4.1 Summary

The statistical analyses using Regional Flood Flow equations (Section 3.2) indicate that the CNB (1977) Province-wide and Acres (1977) approaches provide reasonable estimates of flood flows for annual runoff events. Estimates of the 1:20 and 1:100 year flows at the Water Survey gauge, for example, are as follows:

Regional Approach	Flood Flow Estimate (m^3/s) 1:20 year	1:100 year
CNB Province-wide	42	56
Acres Northern area	34.4	45.4

Monitored streamflow data from snowmelt and rainfall events on Walker Brook and the nearby Eel River enable refinement in th[illegible]e estimates, and provide the following flow statistics:

Streamflow Analyses	Flood Flow Estimate (m^3/s) 1:20 year	1:100 year
Flood flow estimate	35.7	44.2
Upper 95% confidence limit	46.2	62.2

The latter values give a measure of confidence that the 1:20 and 1:100 year flows will be below these upper limits 95% of the time.

Deterministic analyses in Section 3.3 demonstrate that a hydrologic model can also be used to reliably simulate rainfall runoff. The results below summarize the flood flow estimates from 1:20 and 1:100 year return period rainfall events (on average and wet antecedent soils, respectively), and the flood flow expected from an historical storm (Hurricane Belle, 1976) centred over Walker Brook.

	Flood Flow Estimate (m^3/s)	
Rainfall Runoff	1:20 year	1:100 year
Design Rainfall with:		
. dry conditions	15.4	23.8
. wet conditions	30.4	41.7

	Simulated Flood Peak (m^3/s) with:	
Rainfall Runoff	Average Conditions	Wet Conditions
Hurricane Belle	56.2	68.8

Noteworthy in these two tables are that:

- rainfall from the 1:20 year and 1:100 year storms gives flows which are less than the results of statistical analyses (which include spring snowmelt-rainfall events), and
- runoff from the historical storm simulations gives values which are very close to the upper 95% confidence interval of the 1:100 year flood flow (by statistical analysis). The values for average and wet antecedent condition bracket the 95% confidence limit flows within ±9%.

In view of the possibility of a significant hurricane event over Walker Brook during typical soil moisture conditions (AMC II), it is recommended that the peak flow estimates associated with the upper 95% confidence limits (from the statistical analyses in Section 3.2.3) be adopted for the flood damage reduction process.

Table 3-11 summarizes the instantaneous flood flow peaks at various locations in the watershed. These flood flows are employed in the following sections to develop and assess flood damage reduction options.

TABLE 3-11

WALKER BROOK

DESIGN FLOOD FLOWS

LOCATION AND AREA (km^2)	RETURN PERIOD (YEARS) AND FLOW (m^3/s) 1:2	1:20	1:50	1:100
. Walker Bk near mouth (67.4)	27.1	51.7	61.8	69.2
. Walker Bk at WSC gauge (60.8) near Sugarloaf Mt.*	23.8	46.2	55.5	62.2
. Walker Bk u/s of Sugarloaf (45.0)	17.9	33.7	40.4	45.0
. Walker Bk near Val D'Amour (14.0)	5.6	10.3	12.2	13.7

* Flows by transfer from Eel River. Others by the formula:

site flow = (flow at gauge/spring flow at gauge) * (spring flow at site)

where spring flows are those given in Table 3.9.

4.0 HYDRAULIC ANALYSES

4.1 GENERAL

The Hydrotechnical Study of the Walker Brook Flood Plain (SMCC, 1985), and earlier flood information studies, for example, by Folster (1980), identified flood-prone areas along the channel of the brook. Subsequently, Flood Risk Maps were prepared for the Campbellton area, and a Flood Information Map was developed for the Val D'Amour Area (both by Environment Canada and the New Brunswick Department of Environment, 1986). These maps identify significant flood-prone areas at the following general locations:

- near Highway 270 crossing above the school in Val D'Amour;
- a large portion of St. Albert, from Pine Street up to Dover Street and west to the Shopping Mall;
- the area upstream of the lower Dover Street Crossing;
- upstream of Gallant Drive; and
- downstream from Gallant Drive, through Village Avenue to the tidal basin below the dam ruins.

Flood levels at these locations were simulated previously for 1:20 year and 1:100 year return period flows (SMCC, 1985). Since that time, however, new bridges and culverts have been put in place, and the flood flow hydrology has been refined (Chapter 3.0 herein). As a result of these factors, the previously-defined 1:20 year and 1:100 year flood levels have changed.

This section of the report briefly describes the analysis to determine the nature of these changes and the magnitude of the 1:2, 1:20, 1:50 and 1:100 year flood levels in flood-prone areas along Walker Brook.

4.2 BACKWATER MODELLING

In order to determine water levels resulting from existing conditions and from proposed changes to reduce flood damages, it is necessary to hydraulically analyze the passage of flood flows along the channel. This is commonly undertaken using the HEC-2 backwater model.

The HEC-2 model is a computer model which simulates the hydraulic characteristics of a channel and its floodplain, and the bridges, culverts and other features within it. The model is established by using field- measured cross sections, topographic mapping, interpretation of aerial and ground photography, and other site-specific data. This model was prepared for earlier studies, and its development and application are thoroughly described in the Main Report of the Hydrotechnical Study of the Walker Brook Flood Plain (SMCC, 1985).

4.2.1 Application for Flood Damage Reduction

A version of the model was provided for this study in order to maintain consistent results with the earlier work. Certain revisions were required in the previous model, principally to bring it up to date with existing conditions. For example, it was necessary to:

- revise the flood flows (estimated earlier and made more exact in the enclosed hydrologic studies, Chapter 3);
- incorporate the new Gallant Drive Bridge, developed in 1987;
- incorporate changes at culvert/bridge crossings in Val D'Amour; and
- incorporate the proposed Village Avenue Bridge.

Several other changes were made to ensure that the format of the earlier data was consistent with the most up-to-date version of the backwater model. Once these revisions were completed, the model was employed to estimate the 2, 20, 50 and 100 year return period open water flood levels for the existing case. Then it was re-employed to determine the benefits that could be gained from lower floodwater levels that would result from the implementation of various flood damage reduction alternatives (Chapter 6.0).

Tables 4-1 and 4-2, located at the end of this chapter, summarize the updated, computed water surface elevations for various locations along Walker Brook in Campbellton and Val D'Amour, respectively. The section numbers in the left column of these tables correspond to the same sections used in earlier studies and those employed in the Canada-New Brunswick Flood Risk Maps, for Walker Brook, and Flood Information Maps for Val D'Amour. The levels obtained from the earlier studies are given in brackets.

A brief review of the Tables shows that some water levels are higher than the levels estimated earlier (SMCC, 1985), but most current levels are similar or slightly lower. The slight reductions in water levels result from slightly lower flow estimates derived for this study. The large reduction in levels at Gallant Drive, for example, and the slight increase in levels near the Alexander Street footbridge are discussed below.

4.2.1.1 Village Avenue and Gallant Drive. The flood-prone area upstream of the Gallant Drive crossing has been substantially reduced by construction of a new bridge at Gallant Drive in 1987. A similar effect will be realized by the new bridge at Village Avenue (proposed for construction). Hence for purposes of this study, both bridges have been assumed to be in place and their costs and any benefits from their construction have not been factored into this analysis.

4.2.1.2 Alexander Street Footbridge. Water levels at this location are indicative of the overall sensitivity of levels to channel geometry. At this site, and also just below Pine Street, it was considered appropriate to include cross sections in this study which model the flow constriction in these narrow channel reaches. These constrictions raise the water levels locally, but have no significant effect above or below the constrictions.

Other changes which were made to the earlier (SMCC, 1985) backwater model stem from field surveys conducted in January 1988 as part of this study. The noteworthy changes include:

- landfill on the north bank near Matheson Street and the Lily Lake Road Interchange (Section 540,000);

- embankments associated with the new Gallant Drive Bridge and new construction on the southeast corner of Gallant Drive and Esquadich Street (Section 536,800);

- fill along the south bank as part of the Yorsdun Development, between the lower Dover Street crossing and Ila Street (Section 33,100);

- fill along the south bank near the upper Dover Street crossing at the Honda dealership (Section 28,000); and

- culvert replacements/abandonments/washouts in Val D'Amour.

In all, these changes since the earlier study (SMCC, 1985) are reflected in the revised water levels summarized in Table 4-1 and Table 4-2, at the end of this chapter.

4.2.2 St. Albert Spill Area

The St. Albert area is affected by out-of-channel discharges originating from upstream near the Route 11/Route 270 interchange. The source of these flows is likely a combination of:

- runoff from the hillside slopes north and west of St. Albert;

- spill from a small western tributary, originating about 5 km west of the city limits (sub-watershed area #130, Figure 3-6); and

- spill from the main channel just upstream of the Route 11 overpass.

These sources contribute to a "spill" flow which circles around the shopping mall and drive-in theatre, ponds at Dover Street, and then spills across Dover Street and Alexander Street before rejoining the main flow upstream of Pine Street. The 1:20 and 1:100 year flows, identified by SMCC (1985) as being 10.8 m^3/s and 11.3 m^3/s respectively, were revised herein to also include spill from the main branch over Val D'Amour Road and Dover Street. This brings the total spills to 13 m^3/s and 14 m^3/s respectively.

Flood levels and the distribution of flows in passage through St. Albert were scrutinized by examining ponding areas and possible pathways through the area, and by developing a HEC-2 backwater model of the overland flow. The model was initiated at Pine Street and carried upstream over Alexander Street to Comeau Street. At that point the overland flow divides into a section along Boucher Street and another along the Val D'Amour Road Diversion. This flow was recombined at Dover Street and the model was extended westward to an end point in the parking lot of the shopping mall.

For modelling purposes, the spill flow from the parking lot was simulated to cross Dover Street near Boucher Street as a result of culvert blockage/ditch obstruction (assumed complete) at the intersection of Dover Street and the Val D'Amour Road Diversion.

This simulation identified the significance of accurate topographic data and the sensitivity of water levels, and thus the related flood damages, to slight variations in this data. For purposes of this study, the topographic mapping used as a base for the Campbellton Flood Risk Maps was employed to give the path and level of flood flows through St. Albert. Although there are differences between this mapping and spot elevations from some field surveys (e.g. Guitard and Ouelette, 1984), the mapping provides a consistent and adequate set of data over the full study area. Based on a comparison between sources, however, the topographic maps were judged to be reasonably accurate for study purposes.

(For completed HEC-2 backwater modeling output tables, see Appendix B.)

TABLE 4-1

WALKER BROOK - CAMPBELLTON
SUMMARY OF COMPUTED WATER SURFACE ELEVATIONS[1]

Section Number[2]	Location/ Description	Water Level (m) for various Return Periods[3]					
		1:2 yr	1:20 yr		1:50 yr	1:100 yr	
43,210	Outlet at	1.10	1.10	(1.10)[4]	1.10	1.10	(1.10)[4]
543,800	Restigouche River	1.12	1.19	(1.23)	1.26	1.34	(1.52)
543,000		1.14	1.26	(1.32)	1.36	1.44	(1.65)
542,000		1.18	1.41	(1.45)	1.55	1.67	(1.86)
42,800		1.22	1.51	(1.57)	1.67	1.80	(2.01)
42,810	CNR Main Line	1.22	1.51	(1.57)	1.68	1.80	(2.01)
42,820		1.22	1.51	(1.57)	1.68	1.81	(2.01)
42,830		1.22	1.51	(1.58)	1.66	1.79	(2.01)
541,000	Water Main Crossing	1.25	1.58	(1.66)	1.76	1.89	(2.13)
540,000		1.36	1.81	(1.68)	2.01	2.14	(2.15)
539,000		1.37	1.82	(1.75)	2.01	2.15	(2.19)
539,880		1.38	1.82	(1.75)	2.01	2.15	(2.19)
39,000	Old Dam Ruins	2.23	2.81	(2.91)	2.99	3.13	(3.32)
538,000		2.61	2.87	(3.19)	2.93	3.04	(3.49)
537,800		4.54	4.88	(6.15)	5.04	5.15	(6.68)
37,000	Village Avenue[1]	4.78	5.37	(8.19)	5.55	5.68	(8.19)
37,100		5.51	5.78	(8.65)	5.98	6.12	(8.99)
536,800		6.72	6.95	(8.66)	6.98	6.99	(9.00)
36,000	Gallant Drive[1]	6.74	7.02	(8.72)	7.10	7.16	(9.06)
36,100		7.38	7.71	(8.73)	7.87	7.98	(9.07)
36,200		8.04	8.21	(8.76)	8.26	8.29	(9.09)
36,300		8.46	8.71	(8.89)	8.81	8.87	(9.17)
535,000		9.09	9.51	(9.59)	9.65	9.75	(9.98)
35,100		9.69	10.20	(9.99)	10.38	10.50	(10.30)
35,200		10.06	10.50	(10.07)	10.59	10.65	(10.31)
35,800	Gerrard St. Bridge	10.79	11.09	(10.36)	11.19	11.25	(10.45)
35,400		10.86	11.29	(10.77)	11.39	11.46	(10.86)
35,500		11.42	11.70	(11.17)	11.79	11.85	(11.25)
35,600		11.66	12.09	(11.57)	12.19	12.25	(11.66)
534,000		12.23	12.85	(12.95)	12.99	13.04	(13.13)
34,100		12.53	13.16	(13.25)	13.29	13.34	(13.43)
34,800		14.24	15.01	(15.42)	15.01	15.11	(16.76)
34,810	Dover Street	15.65	15.64	(16.99)	15.65	15.69	(16.99)
33,100		15.72	15.89	(17.03)	15.99	16.04	(17.06)
33,200		15.71	15.89	(17.03)	15.98	16.04	(17.06)
533,000	Sugarloaf St. area	16.09	16.68	(17.04)	16.89	17.02	(17.07)
33,500		16.14	16.70	(17.07)	16.90	17.04	(17.13)

TABLE 4-1 (cont'd)

WALKER BROOK - CAMPBELLTON
SUMMARY OF COMPUTED WATER SURFACE ELEVATIONS[1]

Section Number[2]	Location/ Description	Water Level (m) for various Return Periods[3]					
		1:2 yr	1:20 yr		1:50 yr	1:100 yr	
32,100		16.90	17.24	(17.34)[4]	17.35	17.42	(17.56)[4]
32,150		17.29	17.65	(17.73)	17.77	17.85	(18.03)
32,200		17.49	17.83	(18.01)	17.95	18.02	(18.25)
532,000	Old crossing north	18.02	18.40	(18.56)	18.53	18.60	(18.83)
32,300	of Ila St.	-	-	(18.70)	-	-	(18.93)
32,400		-	-	(18.92)	-	-	(19.14)
32,500		19.09	19.56	(19.21)	19.75	19.88	(19.44)
31,100		19.73	20.31	(20.30)	20.50	20.63	(21.13)
531,800	. downstream	20.34	20.73	(21.34)	20.90	21.51	(21.48)
31,200	Pine St.	20.37	22.31	(22.40)	22.38	22.46	(22.51)
30,100		21.66	22.34	(22.41)	22.42	22.49	(22.53)
30,200		-	-	(22.83)	-	-	(23.54)
30,250		21.97	22.51	(23.60)	22.62	22.70	(24.07)
530,800		23.30	23.83	(24.21)	24.45	24.54	(24.89)
30,300	Alexander Street	23.42	24.53	(24.69)	24.79	24.84	(24.88)
30,400		24.01	24.32	(24.65)	24.50	25.09	(25.15)
30,500		24.30	24.82	(24.95)	24.99	25.58	(25.67)
30,600		24.82	25.32	(25.43)	25.50	26.08	(26.16)
30,700		25.10	25.62	(25.73)	25.79	26.38	(26.45)
29,100		25.62	26.38	(26.64)	26.62	26.63	(26.73)
29,200		26.11	26.44	(26.65)	26.54	26.59	(26.75)
29,300		26.61	26.94	(27.01)	27.03	27.06	(27.17)
29,400		27.11	27.44	(27.50)	27.53	27.58	(27.63)
29,800	. downstream	28.68	28.97	(29.45)	29.15	29.25	(29.48)
29,500	. Dover Street	28.74	29.82	(29.55)	30.11	30.11	(29.70)
28,000	. former NBEPC	29.81	30.18	(29.80)	30.17	30.18	(29.95)
528,000		29.81	30.13	(30.24)	30.10	30.08	(30.64)
28,800		31.46	32.86	(33.18)	33.97	34.60	(34.63)
28,100	Highway 11	31.86	32.48	(33.22)	33.46	34.87	(34.79)

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[1] existing conditions, with proposed bridge at Village Avenue and new bridge at Gallant Drive (constructed as design drawings)

[2] cross section numbers are the same as employed in the earlier SMCC (1985) study for defining flood zones along Walker Brook (Table 4.12)

[3] computed water levels consider full flow within the channel, without spill

[4] levels from earlier SMCC (1985) study for 1:20 and 1:100 year conditions

TABLE 4-2

WALKER BROOK - VAL D'AMOUR AREA
SUMMARY OF COMPUTED WATER SURFACE ELEVATIONS

Section Number	Location/ Description	Water Level (m) for various Return Periods^3					
		1:2 yr	1:20 yr		1:50 yr	1:100 yr	
9.860	Private Drive	-	-	(116.87)	-	-	(118.04)
9.300	u/s of Church	117.08	117.39	(117.39)	118.00	118.15	(118.15)
9.370		120.58	120.89	(120.16)	120.99	121.06	(121.66)
9.380		121.08	121.39	(121.66)	121.49	121.56	(122.15)
9.390		121.58	121.89	(122.16)	121.99	122.06	(122.68)
509.000		122.66	122.97	(123.24)	123.07	123.14	(123.65)
9.010		123.32	123.65	(123.95)	123.76	123.84	(124.16)
9.020		123.74	124.07	(124.34)	124.18	124.25	(124.65)
9.030		124.28	124.60	(124.84)	124.70	124.77	(125.16)
9.040		124.75	125.08	(125.34)	125.19	125.27	(125.64)
9.050		125.27	125.58	(125.85)	125.68	125.75	(126.16)
9.060		125.76	126.09	(126.34)	126.19	126.27	(126.63)
9.070		126.25	126.58	(126.84)	126.70	126.77	(127.18)
9.090		126.76	127.08	(126.89)	127.18	127.25	(127.67)
8.800		127.34	127.77	(128.16)	127.92	128.03	(129.12)
8.810		127.71	128.27	(128.17)	128.47	128.64	(129.42)
508.000		127.94	128.53	(129.56)	128.81	129.05	(129.54)
8.010		128.44	128.80	(129.55)	128.92	129.01	(129.51)
8.020		129.10	129.54	(129.48)	129.65	129.72	(129.68)
8.030		129.43	129.68	(129.40)	129.77	129.82	(130.15)
8.040		129.46	129.73	(129.91)	129.81	129.94	(130.47)
8.050		129.45	129.87	(130.39)	130.07	130.23	(130.54)
8.055		129.50	129.83	(130.86)	130.07	130.24	(131.01)
8.820	Private Road	131.24	131.55	(131.84)	131.66	131.75	(132.17)
8.830		131.48	131.87	(132.35)	132.01	132.13	(132.75)
8.100		131.67	132.17	(132.81)	132.35	132.48	(133.42)
8.840		131.69	132.09	(132.96)	132.35	132.55	(133.51)
8.850	Bridge/Body Shop	132.00	132.50	(132.97)	132.62	132.76	(133.66)
8.200		132.12	132.79	(133.51)	133.00	133.17	(133.97)
8.860/8.220		132.16	132.81	(133.97)	133.01	133.17	(134.28)
8.870	Bridge Ruins	132.21	132.95	(134.35)	133.18	133.39	(134.74)
7.100		132.34	132.99	(134.86)	133.22	133.43	(135.36)
7.800		133.90	134.34	(135.42)	134.50	134.62	(135.66)
7.810		134.23	134.53	(136.62)	135.11	135.54	(136.97)
507.000		134.45	135.12	(136.67)	135.56	135.93	(137.03)

5.0 FLOOD DAMAGES

5.1 GENERAL

The damages arising from the flooding of Walker Brook can be recognized as arising from either one, or a combination, of three flow conditions:

- due directly to increased stream flow;
- due to backwaters caused by a structure or structures constricting the flow; or
- arising from a spill over or around a constriction or constrictions or a blockage to flow.

The damages themselves may consist of one or more of the following distinct flooding impacts. First, there is damage to a structure and, if a building, its contents. This in turn may be subdivided to residential dwellings; commercial establishments; industrial facilities; and miscellaneous other buildings such as schools, hospitals, municipal facilities, and funeral homes, to name only a few;

Secondly, there is damage to non-building structures such as culverts and bridges eroded or destroyed, power and telephone poles toppled or requiring resetting, and road surfaces cut.

Thirdly, which relates closely to the second, is the erosion of landscaped surfaces, of stream banks, of ditches and of side hills, including bridge approaches.

Fourthly, there is cost of the clean-up after the flood.

Finally, there is the loss that will occur if a flood impedes or impairs the access of customers to a business, prevents workers from getting to work, or increases the cost of transportation to and from work.

For many flood-prone areas computer programs have been employed to estimate the value of the damages. One example of such a program is the Flood Damage Model developed by Paragon Engineering Limited for the Canada-Ontario Flood Damage Reduction Program. Version 200 of this program provides estimates of damages, broken down by structural type (ie. commercial, industrial, seven residential types and a miscellaneous category), as well as by type of damage (ie. direct, indirect and public).

Unfortunately the computer model cannot be utilized for estimating the flood damages of Walker Brook because the gradients are too steep. However, the estimated damages provided in the program for each residential and non-residential type of structures are valid when adjusted to 1988 values by means of the Consumer Price Index (CPI). The Program provides the residential damages in 1984 dollars, and the commercial, institutional, and miscellaneous structure damages in 1979 dollars. The CPI, provided by the N.B. Statistics Agency, for 1979 was 80.4, for 1984 it was 122.8, and is estimated to be 141.8 for 1988. Consequently the 1988 damage costs were calculated as follows:

. for residential structures,
141.8 (1988 CPI) ÷ 122.8 (1984 CPI) = 115.5%
:. 1.155 x Program dollar estimate; and

. for non-residential structures,
141.8 (1988 CPI) ÷ 80.4 (1979 CPI) = 176.4%
:. 1.764 x Program dollar estimate.

Since the cost of materials and furniture is more expensive in northeastern New Brunswick than it is in Ontario, and while labour costs are lower than in Ontario, it was felt that the costs of repairing/replacing damages would be comparable between the Ontario communities and the Campbellton area. Therefore no further adjustment was made to the figures in the computer program.

A sample of the damage curves for a one storey home with basement is shown in Figure 5.1, which follows.

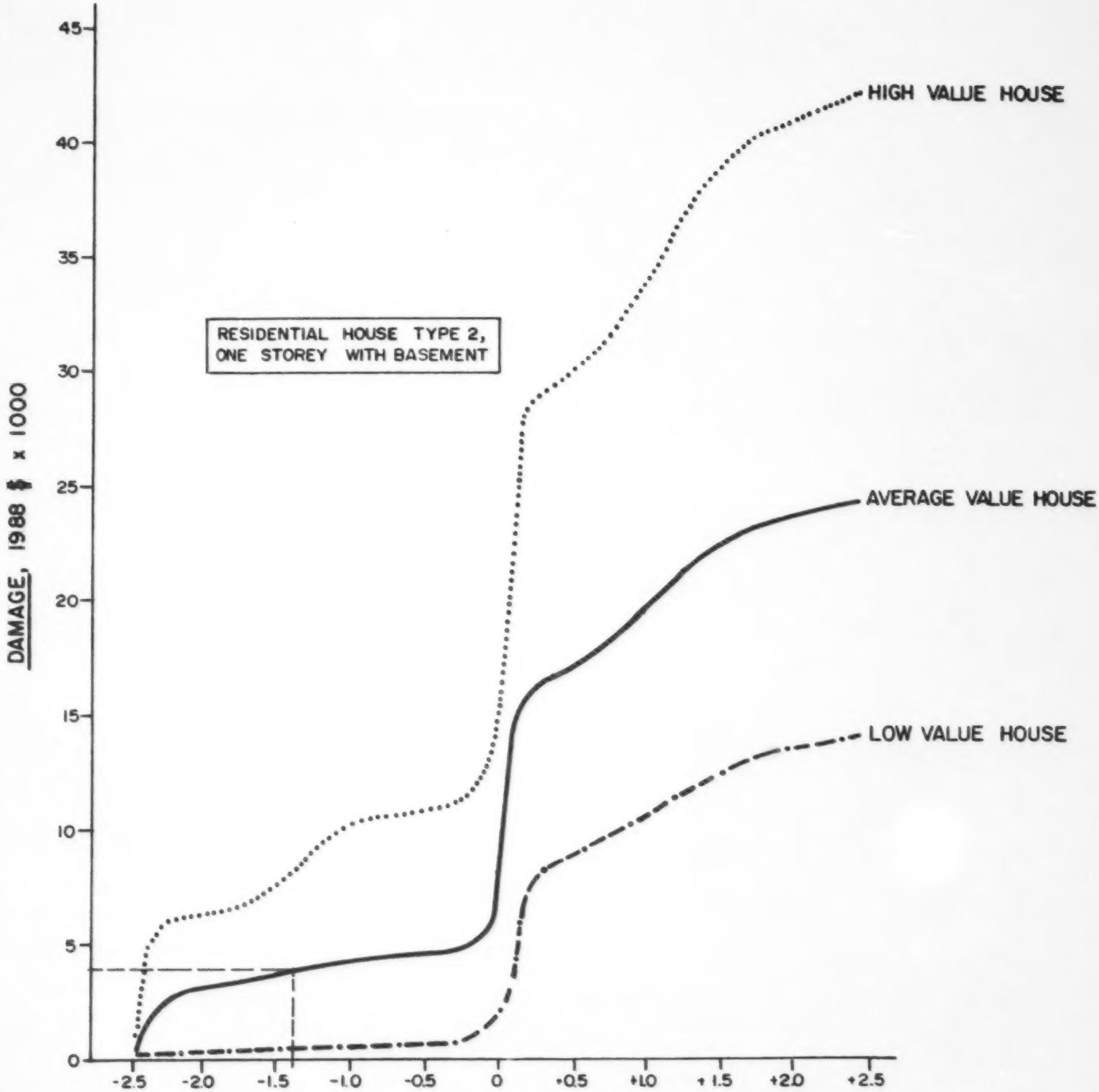
As noted on the preceding page, the gradient of the streamflow caused some problems in estimating the damages. For example, it is obvious that a difference of as little as one centimetre in a basement window level can mean the difference between a flooded and a dry basement; yet, because of the gradient of the flow, the difference between the water level on the upstream side of a residential dwelling with that on the downstream face is frequently in excess of 20 centimetres. Consequently, the first of many assumptions was that the flood entry point was on the upstream side of the structure unless specific data indicated otherwise.

5.2 AFFECTED AREAS

There are two areas in which the study was concentrated; the flood plain from the Route 11 crossing of Walker Brook to its entry into the tidal waters of the Restigouche River, and the community of Val D'Amour.

The areas affected in the Atholville/Campbellton area may be summarized as follows. (See foldout Figure 5.2 at the end of this chapter.)

i) The spill flooding the Sugarloaf Shopping Mall parking lot and arriving via the Val D'Amour highway will occur in conjunction with the 1:50 year and 1:100 year floods. It will not occur at lesser flood levels. Both will crest Dover Street at Boucher Street. In total, an estimated 14 m^3/s are expected to flow into Boucher Street. This will immediately divide, with an estimated 6 m^3/s continuing along Boucher Street and the lots on the southside of the street. The other estimated 8 m^3/s will flow northward to join the flow in the ditch along the Val D'Amour Road



WALKER BROOK

FLOOD PLAIN STUDY PHASE II

SAMPLE DAMAGE CURVES

FENCO FIG. 5-1

Diversion, for a total flow in excess of 9 m^3/s. The two spill streams will re-unite prior to reaching Alexander Street, but divide again with the minor flow returning to the Brook immediately to the east of the intersection of Alexander and Val D'Amour Streets. The major flow, of 12 m^3/s ±, will continue in a north-easterly direction, regaining the ditch along the by-pass until it, in turn, returns to the Brook in the low area to the west of the Pine Street intersection.

The area within the Campbellton City Limit west of Dover Street is zoned as industrial in the City of Campbellton Zoning Map, Schedule "A", as prepared by the Restigouche District Planning Commission, and the area to the east is zoned as residential (ie. that general area between the Val D'Amour Road Diversion and the Dover Street Upper and Lower Crossings of Walker Brook). The area of Atholville abutting the industrial zoning at the west City Limit of Campbellton is commercial, consisting of the Sugarloaf Shopping Mall.

ii) The 1:50 year and 1:100 year backwater upstream of the upper Dover Street crossing will peak at Elev. 30.11. This level will crest Dover Street, both north and south of the crossing and Val D'Amour Road. The modelling forecasts less than 0.25 m^3/s over Dover Street south of the crossing. This flow is expected to follow ditch lines and is not expected to cause any damage. The flow over Val D'Amour Road is expected to peak at 1 m^3/s. This volume is not expected to add more than 0.01 m to the water level of the mall spill flowing into Boucher Street across Dover Street. The majority of the flow (2 m^3/s ±) will be across Dover Street from just south of the crossing to Val D'Amour Street, with the greatest volume returning to the Brook via the ditch on the east side of Dover Street. A lesser volume, not more than 0.2 m^3/s, will flow along Val D'Amour Street and return to the Brook via Belle Isle Street.

Upstream of the crossing the affected area includes Municipal services and storage, NB Power Office, service area and storage, and NB Highway Patrol quarters.

iii) The backwater resulting from the Alexander Street footbridge will affect some out-buildings, but no major structures.

iv) Below the Alexander Street footbridge there is one structure that will be affected by 1:50 year and higher floods as the waters surge after flowing through the constricted bridge opening.

v) The backwater from the existing Pine Street bridge, at flood levels at-and-above the 1:20 year forecast, extends north to Val D'Amour Road Diversion and south to Van Horne Crescent. There is spill over Pine Street north of the crossing, however there are no structures in the path of the spill. The flood levels do not crest over Pine Street south of the crossing. This is a residential area.

vi) At the lower Dover Street crossing of Walker Brook, recent landfills on the south bank have considerably reduced the backwater flooding; however on the north bank, 1:20 year and greater flood levels will approach the intersection of Bornemouth and Ortona Streets. The spill is restricted to the north side of the crossing; however, since there is a downhill slope to Dover Street for some 100 metres beyond the crossing, some of the spill will continue eastward along Dover Street before returning, between the structures, to the Brook.

Structures in this area are limited to residential structures, most of higher than average quality.

In the community of Val D'Amour there are presently no culverts or bridges that appear to restrict the flow sufficiently to cause backwater flooding of structures. One commercial building is, however, in danger of flooding at 1:20 year flood levels, and above. Localized bank erosion is evident, and should a major collapse of a bank cause damming, flooding will certainly result; unfortunately, it is difficult to predict the location of such a collapse.

No study of affected areas was undertaken between the community of Val D'Amour and the entry to Sugarloaf Provincial Park.

5.3 DAMAGES

5.3.1 Damage Estimation

This section provides the methodology that was employed to estimate the value of the damages at each flood recurrence interval.

The elevation of the first floor for all residences and the commercial, industrial and other miscellaneous buildings had to be obtained as a first step. For the area adjacent the lower Dover Street Crossing, and in the community of Val D'Amour, a site survey was conducted. The survey established the ground elevation and the first floor elevation of each building in the 1:100 flood area. Photographs were also taken to provide visual evidence of the quality of the dwellings and the type of the buildings (ie. seven types of residential, and commercial, industrial etc.).

For the areas upstream of Pine Street the ground elevation and first floor elevation of the buildings was available from the Guitard survey.[1] (The elevations of Plaza Pontiac and Sugarloaf Honda, new buildings, had to be surveyed). The houses were photographed to show the quality of the construction and the type of building and, since flood entry levels were not available, to provide a means of estimating the height of the entry level above ground level. At the time of photography all buildings were visually inspected, and where basements were renovated to provide additional living space, this was recorded.

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[1] Guitard, Michel, and Ouelette, John, A Data Collection Report Regarding the St. Albert Flood Plain, February 1984.

A problem was identified with the elevations recorded by Guitard along Boucher Street and Val D'Amour Street between Dover Street and McIntyre Street. His survey of Dover Street between the Val D'Amour Road and Walker Brook does not agree with the topographic contour plan. He shows the intersection of Dover and Boucher Streets 0.5 metre lower than the spot elevation on the plan, and the intersection of Dover and Val D'Amour Street as 0.8 metre lower than the spot elevation on the plan. Since this was part of the closed circuit that included the subsequent house elevations on Boucher Street and Val D'Amour Street, it was concluded that there was also an inconsistency in the house elevations on these two streets. The discrepancy was further complicated by the fact that the elevations by Guitard and those on the plan for Alexander Street were in agreement.

The method used to resolve this dilemma was to prorate, by distance, the 0.8 metre discrepancy at Dover and Val D'Amour Streets to zero difference at Alexander and Val D'Amour Streets. In other words, a house half-way between the two points would have its elevation adjusted up by 0.4 metre to have it concur with the elevations of the contour plan which was the base for the flood flowage delineation. A similar solution was employed to resolve the 0.5 metre difference noted on the west end of Boucher Street.

The new flood levels were plotted on the contour maps for the 1:100, 1:50, 1:20 and 1:2 recurrences based on the new cross sectional data. The water level at the upstream face of each building was extrapolated from the cross section data. The depth of the water, the first floor elevation, and the type of the building structure were then utilized to identify the damage as defined by the updated Paragon program.

In addition to the damages identified in the Paragon Model, certain other direct damages had to be considered. Those damages and the means of assessing them are as follows:

- Clean-up of Flood Transported Debris.
  All floods pick up and transport debris, but waters flooding down a steep gradient carry more material than those on a gentle flood plain. The deposition of the debris will occur in backwaters and in shallows as the flood recedes, but in only a few cases will there be any quantity of debris deposited on any specific property.

  The cost of the clean-up was estimated as the time required to collect the material on the property, and to hire a truck to remove the debris. The costs were estimated on a property-by-property analysis of the points identified above, but in no case did the clean-up cost exceed $150 per residential property. Clean-up costs were considered only when the flood did no damage to the structure.

- Erosion Repair.
  Erosion is the greatest where the flood waters race down steep slopes that have unprotected surfaces. The amount of erosion was related to a composite of the volume of the flow, the degree to which the flow is channeled by natural or man-made obstructions, the steepness of the slope, the type of soil, as well as the ground cover. The costs relate directly to the materials and labour required to restore the site to its original state. Where a residential or other property has received landscaping treatment, such as the dwellings immediately downstream of the lower Dover Street crossing, the costs of the erosion repairs were considerable.

- Damages to Public Works and Structures are all based on the 1979 flood which is assumed to represent a 1:50 year recurrence.[1]

  Estimates for the other flood recurrences are made as follows:
  - that no costs are incurred for 1:2 year floods; and
  - that costs for other flood recurrences will be relative to the volume of their flow (cms) in excess of the 1:2 year flood flow.

  Consequently, if the 1:50 year costs are equal to 100% the 1:100 year costs will be

  (69.2 cms-27.1 cms) ÷ (61.8 cms-27.1 cms) x 100 = 121% of the 1:50 year costs;

  and the 1:20 year costs will be

  (51.7 cms-27.1 cms) ÷ (61.8 cms-27.1 cms) x 100 = 71% of the 1:50 year costs.

  Damages to public works and structures include the reparations paid by the City of Campbellton, the Village of Atholville, the Department of Transportation (DOT), the New Brunswick Electric Power Commission (NB Power), and the New Brunswick Telephone Company (NB Tel).

- Where access to a commercial or industrial structure is denied by floodwaters, either by inundation of the structure or by the flooding of streets, an estimate of loss of business has been made. In all cases the loss has been calculated as $2.50 per square metre per day. In no case was access estimated to be cut for more than one day.

- Where access to the NB Power/Municipal Building is cut off by the floodwaters, it was recognized that certain workers would still have to report for work. These workers would require transportation either by heavy equipment or by boat. The cost of this special transportation requirement has been estimated to cost $500 per day.

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[1] Folster, H.E., Review of Historical Floods Walker Brook and Campbellton Waterfront, August, 1980, quotes on page 4 from local newspaper 1979 as "the worst flood of the past half century". If this is reasonable then 1979 conservatively equates with a 1:50 year flood recurrence.

- Where floodwaters prevent a resident of the area from getting to work, there is a loss of income. In 1986 the Statistics Canada census indicated that the average earned income in Campbellton was $12 600 per year and that there were, on an average, 1.45 wage earners per household. On the basis of 11 statutory holidays and 10 vacation days per year, there are 240 working days per year, therefore the average daily lost income per household can be calculated as: $12 600 x 1.45 ÷ 240 = $76.13 in 1986 dollars.

  The average increase in income between 1986 and 1988 has been 9.1% in New Brunswick, therefore the average Campbellton household earned income per day in 1988 will be approximately $83.05.

  Therefore, where a house is inundated, surrounded by water, or the flooding of nearby streets prevents exit from an area of the community, then there is estimated to be an average daily cost of $83.05 per residence so affected.

5.3.2 Area Specific Damages

5.3.2.1 Community of Val D'Amour. Damages in the community of Val D'Amour are, for the most part, limited to the costs of clean-up, since the undersized private crossings have been destroyed or abandoned and replaced with culverts judged to be capable of handling the 1:100 year flood flow.

Insofar as damage to buildings and their contents are concerned, only two, a service station and a large shed have been identified for potential flooding. If the flood levels are taken at the upstream face of the building versus the downstream wall the effects are very different, as Table 5-1 clearly shows.

TABLE 5-1
FLOOD GRADIENT IMPACT @ SERVICE STATION

	Flood Elevation			Flood Depth on Floor, m	
Return Period	Upstream Wall	Downstream Wall	Floor Elevation	Upstream Wall	Downstream Wall
1:100 yr	124.95	124.77	124.75	0.20	0.02
1:50 yr	124.87	124.70	124.75	0.12	-
1:20 yr	124.76	124.60	124.75	0.01	-

For the purposes of this study we have assumed that the flood entry will be at the garage doors, which are towards the upstream side of the building. The gas tanks have been assumed to be sound and the refill pipe inlets above flood levels, consequently no loss or clean-up costs were attributed to the loss of gasoline.

Damages and clean-up costs have been estimated as follows:

Service Station: building type 9; class F1; floor area @ 189 m^2.
First floor and flood entry elevation = 124.75

1:100 year flood elevation	124.95;	damages	$12 475
1:50 year flood elevation	124.87;	damages	7 030
1:20 year flood elevation	124.76;	damages	585

Large Shed: building type 10; class PWB; floor area @ 63 m^2.
First floor and flood entry elevation = 127.05

1:100 year flood elevation	127.20;	damages	$ 500
1:50 year flood elevation	127.10;	damages	165
1:20 year flood elevation	127.04;	damages	0

The clean-up in the community is based on approximately 30 buildings having debris to clean up at each flood recurrence. The estimated average cost per clean-up for the 1:100, 1:50, 1:20, and 1:2 year recurrences are $90, $80, $65, and $35 respectively. No allowance has been made to cover erosion.

Loss of business at the service station and the tavern would be for one day in the 1:100 and 1:50 year recurrences. The loss of business is estimated to be $190 for the service station and $110 for the tavern.

The total estimated costs of the damages in the community of Val D'Amour are shown in Table 5-2.

TABLE 5-2

FLOOD DAMAGE - VAL D'AMOUR COMMUNITY AREA

Return Period	Damages	Clean-up	Erosion	Commercial	Total
1:100 yr	$13 000	$2 700	-	$300	$16 000
1:50 yr	7 200	2 400	-	300	9 900
1:20 yr	600	2 000	-	-	2 600
1:2 yr	-	1 100	-	-	1 100

5.3.2.2 Mall Spill Area. The mall spill area is the largest single flood area in the Walker Brook flood plain, extending from the Val D'Amour Road Diversion underpass beneath Route 11 to Pine Street, - a distance of 1.2 kilometres. It is, however, the least understood of the spill areas. The flow, once through the underpass, is split by the highway. That on the east side of the road is further divided, with some passing to the south of the Sugarloaf Shopping Mall and returning to the Brook behind the Municipal Garage. The other half of this flow passes into the Mall parking lot and ponds against Dover Street, and crests into Boucher Street.

The flow on the west and north side of Val D'Amour Road Diversion, augmented by local runoff, follows a deep ditch to the junction with Dover Street. At that point the culvert under the road is insufficient for the flow, and it crests the highway. This crest flows into the ditch east of Dover Street, on the south side of the highway.

There has been considerable debate regarding the path of the spill after it crosses Dover Street. The historical account by M.E. Folster (1980) and the 'Summary of Residential Interviews...' by M. Guitard (1984) both indicate that the flow is restricted to the ditch along the Val D'Amour Road Diversion; however, the elevations along Dover Street would appear to prevent all but minor quantities from entering the ditch initially, despite the presence of the culvert beneath Dover Street.

Despite the fact that relatively few structures are inundated by the spill, it occurs simultaneously with the 1:100 and 1:50 year backwaters at the upper Dover Street, Pine Street, and lower Dover Street crossings. Consequently the spill is instrumental in the isolation of the commercial establishments at the mall, and a large number of residences on Boucher Street, Boucher Street Extension, the eastern half of Val D'Amour Street, and McIntyre and Alexander Streets.

The following will provide, in detail, the typical methodology that has been used to estimate the damages from each area-specific spill or backwater flooding.

The first step is to trace the extent of the floodwaters, as defined by hydraulic modelling using the contoured topographic maps of the area (1:1200 scale). The Mall spill was defined by a special run of the model which indicated that flow through the immediate Mall area would result in a flow of 14 to 16 m^3/s over Dover Street. Of this, 14 to 15 m^3/s would flow into Boucher Street, with only very minor quantities flowing north of the houses on Dover Street.

As noted previously, the extremely flat levels require that each house be evaluated for damages individually. Hence, with the cresting over Dover Street at Elev. 29.46, and at Elev. 29.13 some 21 m east on Boucher Street, the flood level at the up-stream face (17 m from Dover Street) of the houses at the corner of Boucher and Dover Streets would be Elev. 29.2. (See 413 Dover Street and 52 Boucher Street of Table 5-3, which follows on pages 5-16 to 5-18 inclusive). This was calculated as follows for the 1:100 year recurrence:

$$\frac{(\text{Elev. } 29.46 - \text{Elev. } 29.13)}{21} \times 4 + \text{Elev. } 29.13 = 29.19, \text{ say Elev. } 29.2$$

Using the house at 413 Dover Street as the example: the ground level was established at Elev. 29.2; the basement windows level was estimated at Elev. 29.5; and the first floor level was estimated to be at Elev. 30.6. With the flood elevation at the ground elevation or above, the Paragon Flood Damage Tables, converted to 1988 $, were used. (See Technical Appendix C for the Damage Curves for all seven types of residences, developed from these tables.)

In Table 5-3 it is shown that the house at 413 Dover Street has a finished basement (fb in column 5), that it is a bungalow (2 in column 6), and that is of average construction (2 in column 7). With the flood level 1.4 metres below the first floor level, using the "average value house" curve for a Type 2 house, in Figure 5-1, indicates a flood damage of $3 935.

For the 1:50 year recurrence the flood levels were estimated to be 0.1 metres lower. Therefore, for 413 Dover Street the 1:50 flood level of Elev. 29.1 will not reach the ground level at the upstream face of the building. Consequently the values from the Paragon Model were not used, but it is obvious that there will be a clean-up to be done on the property since there is a flow of water down the front of the property, along the south side, and circling around the back. The clean-up was estimated to cost $150 in 1988 dollars.

TABLE 5-3

EXAMPLE: MALL SPILL FLOOD DAMAGE ESTIMATION

Val D'Amour Street

						Class	Flood Level		ESTIMATED DAMAGES IN 1988$										
House	Elevation			Type of		of	Elevation		Paragon Model		Clean-up		Erosion		Other Damages			Total Damage	
#	Ground	Entry	1st Floor	Basem't	Bldg.	Bldg.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	Desc.	100 yr.	50 yr.	100 yr.	50 yr.
4+g	22.5	22.6	23.5	fb	2	2	22.46	22.38	$4300	$4210	-	-	-	-	-	-	-	$4300	$4210
6+g	22.2	22.5	23.4	ub	4	2	22.48	22.40	2880	2800	-	-	-	-	-	-	-	2880	2800
8	22.7	23.1	24.1	fb	2	2	22.50	22.42	-	-	$100	$50	$125	$50	fences	$150	$50	375	150
10-12	22.9	23.2	24.4	fb	2	2	22.60	22.55	-	-	75	50	50	-	sheds	50	-	175	50
14	22.6	22.6	23.0	ub	2	2	22.80	22.65	5600	4760	-	-	-	-	-	-	-	5600	4760
16+g	22.6	23.2	23.7	fb	2	2	22.90	22.70	4505	4340	-	-	-	-	-	-	-	4505	4340
21	23.8	24.6	24.6	-	1	2	23.55	23.45	-	-	100	100	75	25	-	-	-	175	125
23	24.5	24.7	25.8	ub	4	2	23.85	23.65	-	-	75	75	25	-	shed	125	50	225	125
25	24.1	24.3	25.1	fb	2	2	24.15	23.95	4400	4175	-	-	-	-	-	-	-	4400	4175
27	24.4	25.1	25.6	ub	4	2	24.35	24.15	-	-	75	75	50	-	fences	100	50	225	125
29	24.4	24.8	25.3	ub	4	2	24.65	24.50	3025	2945	-	-	-	-	-	-	-	3025	2945
31	24.5	25.9	25.9	-	3	2	24.90	24.70	-	-	75	75	50	-	fences	100	75	225	150
33	24.6	24.9	25.4	ub	4	2	25.05	24.85	3180	3025	-	-	-	-	-	-	-	3180	3025
35	24.8	24.9	25.4	ub	4	2	25.20	25.00	3920	3155	-	-	-	-	-	-	-	3920	3155
39	25.2	25.5	25.5	-	3	1	25.45	25.20	1180	-	-	75	-	25	-	-	-	1180	100
39½	25.2	25.5	25.5	-	3	1	25.48	25.23	1555	-	-	75	-	25	-	-	-	1555	100
41	25.2	25.4	25.4	-	9	E1*	25.55	25.25	3815	-	-	75	-	50	-	-	-	3815	125
43	25.2	25.7	25.7	-	1	1	25.60	25.30	-	-	100	75	75	50	fences	150	75	325	200
45	25.9	26.2	26.7	ub	2	2	25.65	25.35	-	-	75	50	50	25	-	-	-	125	75
63	28.7	28.8	29.2	fb	2	2	27.90	27.80	-	-	50	50	-	-	fences	75	25	125	75
65	28.8	28.9	29.4	fb	2	2	28.05	27.90	-	-	75	50	-	-	fences	75	25	150	75
67	28.8	29.4	29.4	-	1	2	28.15	28.00	-	-	75	50	-	-	fences	75	25	150	75
69	29.0	29.3	29.8	fb	2	2	28.25	28.10	-	-	100	75	25	-	fences	75	25	200	100
TOTAL ESTIMATED DAMAGES, VAL D'AMOUR STREET									38360	29410	975	1000	525	250		975	400	40835	31060

*repair shop
@ area = 82 m^2

LEGEND:

g = garage (detached)
fb = finished basement
ub = unfinished basement

Type of bldg. as per Paragon Model classification; i.e.

1 = one storey without basement
2 = one storey with basement
3 = two storeys without basement
4 = two storeys with basement
7 = mobile homes
8 = commercial
9 = commercial
10 = commercial

Class of bldg.;

1 = low value
2 = average value
3 = high value

TABLE 5-3 (cont'd)

EXAMPLE: MALL SPILL FLOOD DAMAGE ESTIMATION

Boucher Street

House	Elevation			Type of		Class of	Flood Level Elevation		ESTIMATED DAMAGES IN 1988$											
									Paragon Model		Clean-up		Erosion		Other Damages			Total Damage		
#	Ground	Entry	1st Floor	Basem't	Bldg.	Bldg.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	Desc.	100 yr.	50 yr.	100 yr.	50 yr.	
25	26.1	26.6	27.2	ub	2	2	25.9	25.8	-	-	$100	$100	$150	$100	-	-	-	$250	$200	
27	26.7	27.2	27.8	fb	2	2	25.8	25.7	-	-	75	75	25	25	-	-	-	100	100	
29	27.3	27.9	28.5	ub	2	2	25.9	25.8	-	-	75	75	25	25	-	-	-	100	100	
30	27.0	27.2	27.9	fb	2	2	26.3	26.2	-	-	100	75	100	75	-	-	-	200	150	
31	27.2	27.8	28.4	fb	2	2	26.1	26.0	-	-	125	100	50	50	-	-	-	175	150	
32	27.0	27.2	27.7	fb	2	2	26.6	26.5	-	-	75	75	25	25	-	-	-	100	100	
33	27.2	27.7	28.3	fb	2	2	26.5	26.4	-	-	100	100	-	-	-	-	-	100	100	
34	27.0	27.5	28.0	fb	2	2	26.8	26.7	-	-	100	100	25	-	-	-	-	125	100	
35	27.1	27.5	27.5	-	7	2	27.0	26.9	-	-	100	100	25	-	-	-	-	125	100	
36	27.4	27.9	27.9	-	3	2	27.0	26.9	-	-	125	100	25	-	-	-	-	150	100	
37	27.1	27.5	27.5	-	7	2	26.6	26.5	-	-	50	50	-	-	-	-	-	50	50	
39	27.3	27.6	27.6	-	7	2	26.6	26.5	-	-	-	-	-	-	-	-	-	-	-	
41	27.2	27.5	27.5	-	7	2	26.6	26.5	-	-	75	75	50	-	-	-	-	125	75	
42	27.9	27.9	28.1	fb	2	2	27.4	27.3	-	-	100	75	50	25	fences	$100	$100	250	200	
43	27.3	27.7	27.7	-	7	2	26.7	26.6	-	-	75	50	25	-	-	-	-	100	50	
43a	27.1	27.4	27.4	-	8	El*	27.0	26.9	-	-	75	50	-	-	-	-	-	75	50	*office @ area = 17 m^2
44	27.9	28.2	29.0	fb	2	2	27.8	27.6	-	-	125	100	50	25	fences	75	50	250	175	
45	27.3	27.7	27.7	-	7	2	27.1	27.0	-	-	75	50	-	-	-	-	-	75	50	
46	28.2	28.5	29.2	ub	8	El*	28.1	28.0	-	-	100	100	50	25	shed & fences	100	75	250	200	*hair salon @ area = 56 m^2
47	27.5	28.0	28.0	-	7	2	27.2	27.1	-	-	50	-	-	-	-	-	-	50	-	
48	28.6	29.2	29.7	ub	2	2	28.4	28.3	-	-	100	100	100	75	garage & fences	150	100	350	275	
49	27.7	28.1	28.1	-	7	2	27.5	27.4	-	-	100	75	25	-	-	-	-	125	75	
50	28.8	29.1	29.1	fb	2	2	28.7	28.6	-	-	125	100	100	75	shed & fences	125	100	350	275	
51	27.6	28.0	28.0	-	7	2	27.5	27.4	-	-	-	-	-	-	-	-	-	-	-	
52	29.2	29.7	29.7	-	1	2	29.2	29.1	-	-	150	125	250	200	-	-	-	400	325	
53	27.5	27.9	27.9	-	7	2	27.2	27.1	-	-	75	50	-	-	-	-	-	75	50	
55	28.1	28.5	29.0	fb	2	2	27.6	27.5	-	-	100	50	-	-	-	-	-	100	50	
57	28.5	28.8	29.3	fb	2	2	27.7	27.6	-	-	125	100	-	-	-	-	-	125	100	
59	28.2	28.6	29.1	fb	2	2	27.9	27.8	-	-	150	125	125	100	fences	150	150	425	375	
61	28.4	28.7	29.3	fb	2	2	28.3	28.2	-	-	150	125	75	50	shed & fences	125	125	350	300	
63	28.4	28.8	30.0	fb	2	2	28.6	28.5	$3950	$3810	-	-	-	-	shed & fences	-	-	3950	3810	
65	28.6	29.0	29.0	-	1	2	28.9	28.8	1475	830	-	-	-	-	-	-	-	1475	830	
TOTAL ESTIMATED DAMAGES, BOUCER STREET									5425	4640	2775	2300	1350	875		825	700	10375	8515	

For Legend, see first page oftable

TABLE 5-3 (cont'd)

EXAMPLE: MALL SPILL FLOOD DAMAGE ESTIMATION

House	Elevation			Type of		Class of	Flood Level Elevation		ESTIMATED DAMAGES IN 1988$ Paragon Model		Clean-up		Erosion		Other Damages			Total Damage		
#	Ground	Entry	1st Floor	Basem't	Bldg.	Bldg.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	100 yr.	50 yr.	Desc.	100 yr.	50 yr.	100 yr.	50 yr.	
Boucher Street Extension																				
1	24.8	25.3	25.9	fb	2	2	24.8	24.6	-	-	$100	$75	$50	$25	-	-	-	$150	$100	
2	24.7	25.0	25.0	-	8	Fl*	24.4	24.3	-	-	100	75	-	-	-	-	-	100	75	*area = 140 m^2
8	24.8	24.8	24.3	-	9	El*	24.9	24.8	$5790	$4805	-	-	-	-	-	-	-	5790	4805	*area = 40 m^2
10	25.3	25.6	26.9	ub	8	Fl*	25.3	25.2	-	-	100	75	75	75	-	-	-	175	150	*area = 120 m^2
TOTAL ESTIMATED DAMAGES, BOUCHER STREET EXTENSION									5790	4805	300	225	125	100		-	-	6215	5130	
Alexander Street																				
38	24.8	25.3	25.8	fb	2	2	24.9	24.8	4445	4340	-	-	-	-	-	-	-	4445	4340	
47	25.1	25.4	25.4	-	10	?*	25.6	25.5	9150	5100	-	-	-	-	-	-	-	9150	5100	*area = 410 m^2
TOTAL ESTIMATED DAMAGES, ALEXANDER STREET									13595	9440	-	-	-	-		-	-	13595	9440	
MacIntyre Street																				
2	26.2	26.8	27.4	ub	2	2	25.9	25.8	-	-	125	100	150	125	fences	$150	$100	425	325	
4	26.1	26.5	27.0	fb	2	2	25.7	25.5	-	-	100	50	75	-	-	-	-	175	50	
TOTAL ESTIMATED DAMAGES, MACINTYRE STREET											225	150	225	125		150	100	600	375	
Dover Street																				
413	29.2	29.5	30.6	fb	2	2	29.2	29.1	3935	-	-	150	-	150	fences	-	75	3935	375	
415	29.7	30.2	30.9	fb	2	2	29.2	29.1	-	-	150	150	125	100	-	-	-	275	250	
417	29.6	29.8	30.2	fb	2	2	29.2	29.1	-	-	125	100	150	125	-	-	-	275	225	
TOTAL ESTIMATED DAMAGES, DOVER STREET									3935	-	275	400	275	375		-	75	4485	850	
Sugarloaf Shopping Mall Area																				
Mall	31.5	31.7	31.7	-	8	Fl*	30.3	30.2	-	-	1000	750	-	-	-	-	-	1000	750	*area = 15 915 m^2
Plaza Pont.	29.3	29.5	29.5	-	8	Fl*	29.6	29.6	17500	15250	-	-	-	-	-	-	-	17500	15250	*area = 565 m^2
D&L Elect.	29.6	29.7	29.7	-	9	Fl*	29.5	29.5	-	-	100	75	-	-	-	-	-	100	75	*area = 112 m^2
Sharp	31.4	31.9	31.9	-	9	Fl*	29.7	29.6	-	-	350	100	-	-	fences	150	50	500	150	*area = 574 m^2
Drive-in	31.6	32.0	32.0	-	10	?	31.5	31.4	-	-	1500	1000	1000	500	fences	500	300	3000	1800	
TOTAL ESTIMATED DAMAGES, SUGARLOAF SHOPPING MALL AREA									17500	15250	2950	1925	1000	500		650	350	22100	18025	
TOTALS, MALL SPILL AREA									$84605	$63545	$7500	$6000	$3500	$2225		$2600	$1625	$98205	$73395	

For Legend, see first page of table

Because of the funnelling effect into Boucher Street of all the flood cresting Dover Street, this study has estimated that erosion damages to the property will total $150, in 1988 dollars.

It is estimated that the fence along the south side of the property will receive damages from the floodwaters and its debris, to a limited degree, in the 1:50 year flood recurrence. This has been estimated conservatively at $75, in 1988 dollars.

As shown in Table 5-3, each property impacted by the Mall spill was individually appraised relative to the Paragon Model damage curves where the floodwaters reached the level of the ground at the building foundation; and where they did not reach the foundation but were on the property, an estimate was made of the cost of the clean-up after the flood, the repair of erosion damages, and the repair to fencing and sheds on the property.

In addition to the damages identified above, there are the losses arising from loss of business, both retail sales, industrial sales, and service industry sales. As indicated in the previous section, Mall business loss is estimated at $2.50 per square metre per day, therefore with the Mall spill affecting 11 commercial/industrial properties with a total of 18 031 m^2, the loss in business will be $45 075. This is the same for both the 1:100 and 1:50 year recurrences.

Not included in these calculations are loss of income for individuals unable to get to work, or the damage to utilities or public works. These can only be estimated relative to the total impact of the flood and then be prorated. Table 5.4 provides a summary of the foregoing calculated damages from the Mall Spill. Tables 5.5 through 5.8 have been estimated in a similar manner for the specific areas noted.

TABLE 5-4
FLOOD DAMAGE - MALL SPILL AREA

Return Period	Paragon Damages	Clean-up	Erosion & Misc.	Commercial	Total
1:100 yr	$84 605	$7 500	$6 100	$45 075	$143 280
1:50 yr	63 545	6 000	3 850	45 075	118 470

5.3.2.3 Upper Dover Street Backwater and Spill Area. Due to the less than adequate capacity of the pipe-arch culvert at the upper Dover Street crossing of Walker Brook, backwater elevations will crest over both Dover Street and Val D'Amour Street west of Dover Street in a 1:50 year recurrence. The 1:20 year recurrence will crest over Dover Street but is not expected to flow over Val D'Amour Street into the Mall area.

There are two buildings that will be affected by the backwaters: the first, the so-called NB Power building, which houses the City Works Garage and NB Power (also the N.B. Highway Patrol until January 1989), and has two floors. The office floor at Elev. 31.0 is above the floodwaters, but the service/garage floor at Elev. 29.5, and the basement which is lower still, will be inundated even at the 1:2 year flood levels. The second building, a storage building, is also at the Elev. 29.5 entrance level; but, being further downstream, it will not be flooded by the 1:2 year recurrence flood.

The floor area of the services bays and basement of the NB Power building is approximately 2500 m^2. The 1:2 year flood will place 0.3 m of water depth in the building, the 1:20 and above recurrence will inundate the service area to a depth of 0.7 m. Consequently, the damage has been estimated at $76 600 for the higher depths and $48 225 for the 1:2 year recurrence.

The floor area of the storage building is approximately 140 m^2. It will experience water depths of 0.6 m for the 1:100 and 1:50 year recurrences and of 0.4 m for the 1:20 year recurrence. This is estimated to incur $3 980 of damages for the higher levels and $3 020 damages for the 1:20 year flood level.

The flooding of Val D'Amour Road Diversion west of Dover Street will prevent workers from getting to work without special transportation. The cost of the special transportation has been estimated at $500 per day for the 1:50 and 1:100 flood levels.

It has been estimated that the 1:100 and 1:50 year flooding will both produce approximately 1 m^3/s over Val D'Amour Street. This, if added to the upstream spill into the Mall (Mall spill), will only increase the crest into Boucher Street by approximately one centimetre. If it should happen without the simultaneous occurrence of the upstream spill, it will create no problem, being absorbed into the ditch on the north side of the parking lot.

The spill over Dover Street is estimated to total 2 m^3/s for all floods above the 1:20 year recurrence. Most of this spill will be carried off by the ditches along the northeast side of the road. In the 1:50 and 1:100 year flood levels there will be a minor flow down Val D'Amour Street to Belle Isle Street where it will return to the Brook. Damage from the spill will include clean-up and minor erosion repairs.

The estimated damages arising from the upper Dover Street backwater and spill are shown in Table 5-5, which follows.

TABLE 5-5
FLOOD DAMAGE - UPPER DOVER STREET BACKWATER AND SPILL AREA

Return Period	Damages	Clean-up	Erosion	Miscellaneous	Total
1:100 yr	$80 580	$475	$325	$500	$81 880
1:50 yr	80 580	375	150	500	81 605
1:20 yr	79 670	100	50	-	79 820
1:2 yr	48 225	150	-	-	48 375

5.3.2.4 Alexander Street Backwater and Spill Area. Neither the backwater nor the spill extend any distance from the footbridge. The Paragon Model was applicable to two buildings, - a bungalow without a basement at 44 Alexander Street and a bungalow with an unfinished basement at 46 Mountain View Place. The damages for the 1:100, 1:50, and 1:20 year flood levels are estimated at $10 230, $10 170, and $5 575 respectively.

Clean-up will be required on 12 residential properties and one commercial property in the 1:100 year flooding, and on 7 properties in the 1:50 year, 5 in the 1:20, and 3 in the 1:2 year recurrences.

Erosion will be a problem on 5 properties, and fence repairs also on 5 properties.

Loss of business will impact on the single commercial building, which has a floor area of 147 m^2. This will result in a one day loss of business of $370 in the 1:100 and 1:50 year floods. The damages shown in Table 5-6 represent the estimated cost of the flooding in this area.

TABLE 5-6
FLOOD DAMAGE - ALEXANDER STREET BACKWATER AND SPILL AREA

Return Period	Damages	Clean-Up	Fences & Erosion	Miscellaneous	Total
1:100 yr	$10 230	$400	$600	$370	$11 600
1:50 yr	10 170	275	400	370	11 215
1:20 yr	5 575	125	275	-	5 975
1:2 yr	-	75	125	-	200

5.3.2.5 Pine Street Backwater and Spill Area. The backwater resulting from the limited capacity of the pipe-arch culvert at Pine Street extends some 225 metres upstream, and northeast as much as 180 metres to Arran Street (the start of the Val D'Amour Road Diversion, on Route 270 to the community of Val D'Amour). Consequently, it floods much of the same area as do the waters from the tail of the Mall spill. The damages arising along Val D'Amour Street, in particular those properties between and including number 4 and number 16, are common to both for the 1:100 and 1:50 year flood recurrences.

Other than four of those dwellings identified, the backwaters do not inundate any residential or commercial structures, however two of those residences are flooded in the 1:20 year backwaters. Other than residential structures there are six buildings, garages or sheds, that are inundated by the flood waters. These damages are included with those of the residential property on which they are located.

Clean-up will be required on three residential properties affected by both the Mall spill and the Pine Street backwater. In addition to these properties there will be six others on the north side of the Brook and ten on the south side which will be affected by the

1:100, 1:50 and 1:20 year recurrences. In the 1:2 year recurrence, clean-up will be limited on these properties.

Erosion damage can be expected on all properties affected by the Mall spill and the Pine Street backwater, by three other properties on the north side of the Brook and by another three on the south side for the same flood recurrences.

Damages to fences will be restricted to properties on the north side of the Brook. Six properties, other than those whose damage was calculated by the Paragon flood damage curves, will have fencing that will be damaged by the 1:100, 1:50 and 1:20 year flood recurrences.

The Pine Street spill is restricted to the north side of the Brook. There are no buildings in this area and the floodwaters return to the Brook without damage to any private property.

Table 5-7, which follows, shows the estimated flood damage for the Pine Street backwater and spill areas.

TABLE 5-7
FLOOD DAMAGE - PINE STREET BACKWATER AND SPILL AREA

Return Period	Damages	Clean-Up	Fences & Erosion	Miscellaneous	Total
1:100 yr	$11 580*	$2 600*	$1 700*	-	$15 880*
1:50 yr	9 510*	2 600*	1 600*	-	13 710*
1:20 yr	8 950	2 300	900	-	12 150
1:2 yr	-	1 000	-	-	1 000

*The damages that are common to both the Mall spill and the Pine Street backwater are as follows:

Return Period	Damages	Clean-up	Fences & Erosion	Miscellaneous	Total
1:100 yr	$11 580	$450	$500	-	$12 530
1:50 yr	9 510	450	500	-	10 460
1:20 yr	-	-	-	-	-
1:2 yr	-	-	-	-	-

5.3.2.6 Lower Dover Street Backwater and Spill Area. The backwater rising behind the lower Dover Street crossing of Walker Brook can be expected to cause inundation problems for only one building, a recently constructed split entry bungalow immediately to the northeast of the culvert crossing. Aside from this unit, clean-up, erosion and fencing repairs may be expected at the first property on Dover Street upstream of the crossing, on two properties on Aspen Court, on two properties on Bornemouth Street, and to one property on George Street.

The spill over Dover and Ortona Streets will flow down into a low area of Dover Street that extends the spill some 130 metres beyond the stream crossing. Consequent damages due to inundation will occur at six dwellings with the 1:100 year flooding, five dwellings with the 1:50 year and at four residences at the 1:20 year flood levels.

The estimated flood damages from the backwater and spill arising from the culvert crossing of Dover Street is provided in Table 5-8.

TABLE 5-8

FLOOD DAMAGE - LOWER DOVER STREET BACKWATER AND SPILL AREA

Return Period	Damages	Clean-Up	Fences & Erosion	Miscellaneous	Total
1:100 yr	$28 555	$ 600	$ 200	-	$29 355
1:50 yr	23 850	575	200	-	24 625
1:20 yr	19 115	350	150	-	19 615
1:2 yr	4 230	450	400	-	5 080

5.3.2.7 Gerrard Street and Downstream Crossings. As noted previously, there are no damages to structures at or below Gerrard Street.

5.3.3 General Area Direct Damages

In this section damages are considered that are identifiable only in area-wide terms. They will all be based on the 1979 flood, and consequently represent the 1:50 year flood recurrence. Three types of data are employed. The first is cost information that is verifiable from existing records, the second are costs which are estimates provided by the department which paid for them, and the third are estimates prepared by this study when no other source of information was found.

5.3.3.1 City of Campbellton. The records of the Emergency Measures Organization (EMO) indicate that the City of Campbellton received a payment for 1979 flood reparations in the amount of $64 000. In terms of 1988$, this would equate to $136 600, $112 900 and $80 200 for 1:100, 1:50 and 1:20 flood recurrences respectively.

5.3.3.2 Village of Atholville. The EMO reports that Atholville received $2 495 for reparations following the 1979 flood. This represents, in 1988$, $5 300 and $4 400 for 1:100 and 1:50 flood recurrences respectively.

5.3.3.3 Department of Transportation. The records of EMO indiate that the Department of Transportation (DOT) in Restigouche County received $37 822 from them following the 1979 Walker Brook flooding. The DOT District Office reported that the 1979 Walker Brook flood repair costs were $42 000. Detailed breakdown estimates are no longer available in either case; however, it is known that some expenditures were required for culvert repairs in the community of Val D'Amour, some for road and ditch repair on Route 270 between Sugarloaf Park and the Sugarloaf Shopping Mall, some along Route 11 for repairs to embankments eroded by the diversion of Walker Brook, and some for the restoration of the ditch on the south side of the highway in which the diversion was channelled.

A reasonable estimate of the breakdown of the costs incurred in repairing the damage caused in the community of Val D'Amour, along Route 270 and its interchange with Route 11, and the Walker Brook diversion along Route 11 would be as follows (in 1979 $):

- Community of Val D'Amour - $ 4 000
- Route 270 - 11 000
- Route 11 - 9 000

The flood damage recurrence costs in 1988$ are:

Location	1:100 yr	1:50 yr	1:20 yr
Val D'Amour	$ 8 540	$ 7 055	$ 5 010
Route 270	23 580	19 405	-
Route 11	19 210	15 875	$11 270
Total	$51 330	$42 335	$16 280

5.3.3.4 NB Power. NB Power had to replace a number of poles, transformers and guy wires as a result of the 1979 flooding by Walker Brook. In addition, their storage yard to the west of their depot was inundated and a considerable clean-up was required. The costs to NB Power has been estimated by this study as $6 500 in 1979$.

The corresponding flood damage recurrence costs in 1988$ are:

1:100 yr - $13 875
1:50 yr - 11 465
1:20 yr - 8 140
1:2 yr - -

5.3.3.5 NB Tel. The re-establishment of poles and guys by NB Tel has been estimated by this study to be 50% of the costs incurred by NB Power.

5.3.3.6 General Area Summary. The costs in 1988$ of the direct damages not attributable to a specific area are shown in Table 5-9.

TABLE 5-9
GENERAL AREA DIRECT FLOOD DAMAGES

Authority	Return Period 1:100 yr	1:50 yr	1:20 yr	1:2 yr
Campbellton	$136 600	$112 900	$80 200	-
Atholville	5 300	4 400	-	-
NB DOT	51 330	42 335	16 280	-
NB Power	13 875	11 465	8 140	-
NB Tel	6 940	5 735	4 070	-

5.3.4 General Area Indirect Damages

Indirect damages are losses of income incurred by individuals or businesses when floodwaters cut access to or from the residence of the individual or the place of business. (Although it may also be argued the employer also loses income when an employee is unable to get to work, these indirect damages have not been included.)

An example of this occurs with the 1:20 flood recurrence, and above, when Walker Brook spills over Dover Street at both its upper and lower crossings, over the Pine Street crossing, and over the Alexander Street footbridge crossing. This effectively cuts all access to the area south of Walker Brook between the two crossings of the Brook by Dover Street.

With the 1:50 and 1:100 year flood levels, an additional area lying between Route 270 and Walker Brook, and extending from Pine Street to the Route 11 interchange, is also cutoff from outside access.

The numbers of residences and commercial buildings that can be subject to access severance as a result of Walker Brook flooding are shown in Table 5-10, which follows.

TABLE 5-10
NUMBER OF RESIDENCES AND COMMERCIAL BUILDINGS ISOLATED BY WALKER BROOK FLOODWATERS

	Return Period (years)			
	1:100 & 1:50		1:20	
	Residential	Commercial	Residential	Commercial
Alexander Street(1)	3	1	2	-
Aspen Court	4	-	4	-
Aspen Drive	4	-	4	-
Belle Isle Street	12	-	-	-
Boucher Street(1)	30	2	-	-
Boucher Street Extension(1)	1	3	-	-
Bornemouth Street	2	-	-	-
Dover Street(2)	107	6	98	6
Ila Street	12	-	12	-
John Street	21	-	21	-
Kay Street	26	-	26	-
Kimberly Court	7	-	7	-
MacIntyre (Cormier) Street	2	-	-	-
Mountain View Drive	40	-	40	-
Pine Street	21	-	21	-
Val D'Amour Street(1)	52	4	-	-
Van Horne Crescent	29	-	29	-
Walker Crescent	22	-	22	-
West off Pine Street	11	-	11	-
West of Upper Dover Street(1)	-	9	-	1
Yorsdun Court	3	-	3	-
Total	409	25	300	7

---

(1) Loss of Commercial revenues already included in specific area miscellaneous damages.

(2) Includes 70 apartments at Sugarloaf Towers.

5.3.4.1 Personal Indirect. The flood levels that cut travel from the place of residence to the work place of the individual have been calculated to last only one working day. Therefore, with an estimated daily indirect cost per residence due to lost wages of \$83.05 (as per page 5-11), the total estimated loss per flooding is \$24 915 for the 1:20 year flood recurrence, and \$33 970 for the 1:50 and 1:100 year recurrences.

5.3.4.2 Commercial Indirect. As noted in Table 5-10, the majority of the indirect commercial damages were allocated to a specific area. The six that were not so defined are all located along Dover Street between the two Brook crossings. The total ground floor area of these commercial structures has been estimated to be 2 900 square metres. At an estimated loss of business cost of \$2.50 per m^2 per day for a one day period the indirect damages would total \$7 250 for the 1:100, 1:50, and 1:20 flood recurrences.

5.3.5 Summary of Damages

This section provides the total estimated cost of damages, in 1988\$, arising from the four flood return periods reviewed by this study. Care was taken not to double count any of the damages, in particular those arising in the 1:100 and 1:50 year return periods at the junction of the Mall spill with the Pine Street backwater.

The Paragon methodology and data were utilized in all calculations, and the results have been summarized in Table 5-11, which follows.

TABLE 5-11
FLOOD DAMAGES SUMMARY
(1988 $)

CAMPBELLTON/ATHOLVILLE AREA

	Return Period (years)			
	1:2	1:20	1:50	1:100
Specific Area Damages				
Mall Spill Area	-	-	118 470	143 280
Upper Dover Street Backwater and Spill Area	48 375	79 820	81 605	81 880
Alexander Street Backwater and Spill Area	200	5 975	11 215	11 600
Pine Street Backwater and Spill Area	1 000	12 150	3 250(1)	3 350(1)
Lower Dover Street Backwater and Spill Area	5 080	19 615	24 625	29 355
General Area Damages				
City of Campbellton	-	80 200	112 900	136 600
Atholville	-	-	4 400	5 300
NB DOT	-	11 270	35 280	42 790
NB Power	-	8 140	11 465	13 875
NB Tel	-	4 070	5 735	6 940
Indirect Personal	-	24 915	33 970	33 970
Indirect Commercial	-	7 250	7 250	7 250
Total	54 655	253 405	450 165	516 190

****

COMMUNITY OF VAL D'AMOUR

	1:2	1:20	1:50	1:100
Specific Area Damages	1 100	2 600	9 900	16 000
General Area Damages	-	5 010	7 055	8 540
Total	1 100	7 610	16 955	24 540

---

(1) See note Table 5-7.

5.4 PRESENT VALUE

The present value of estimated damage is the sum of the expected annual cost for a specified period reduced by a defined discount rate.

5.4.1 Expected Annual Damage (E.A.D.)

The expected annual damage is defined as the value of the damages that can be expected on an annual basis when averaged over an extended period of time. The procedure to calculate the E.A.D. is as follows:

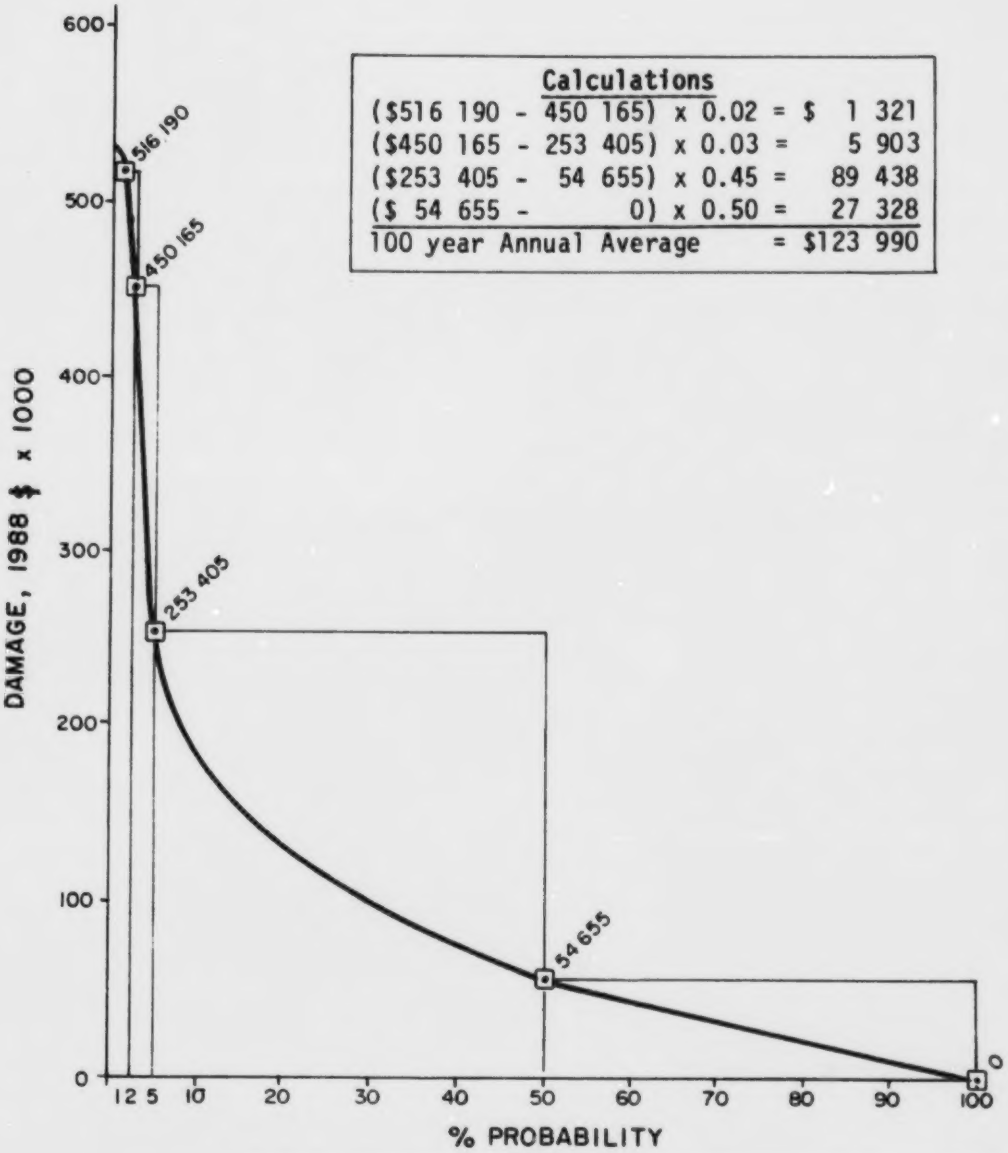
E.A.D. = The sum of the incremental damages when multiplied by their individual probabilities.

For Walker Brook, the damages have been calculated for the 1:2, 1:20, 1:50, and 1:100 flood recurrences. The damages for the period less than 1:2, which would be 1:1, are considered as zero.

The probability of a 1:100 year flood = 1%, of a 1:50 = 2%, a 1:20 = 5%, a 1:2 = 50%, and a 1:1 would have a 100% probability.

The flood event damages/probabilities relationships for the subject areas may be presented in graphical form, with the X axis (horizontal) representing % Probability and the Y axis (vertical) representing the Damages costs.

In the following two examples (Figures 5-3 & 5-4), the curves and calculations have been provided for the two study areas. The increments of damage are multiplied by the increments of probability and the products thereof are totalled to provide the expected annual damage over a 100 year period.



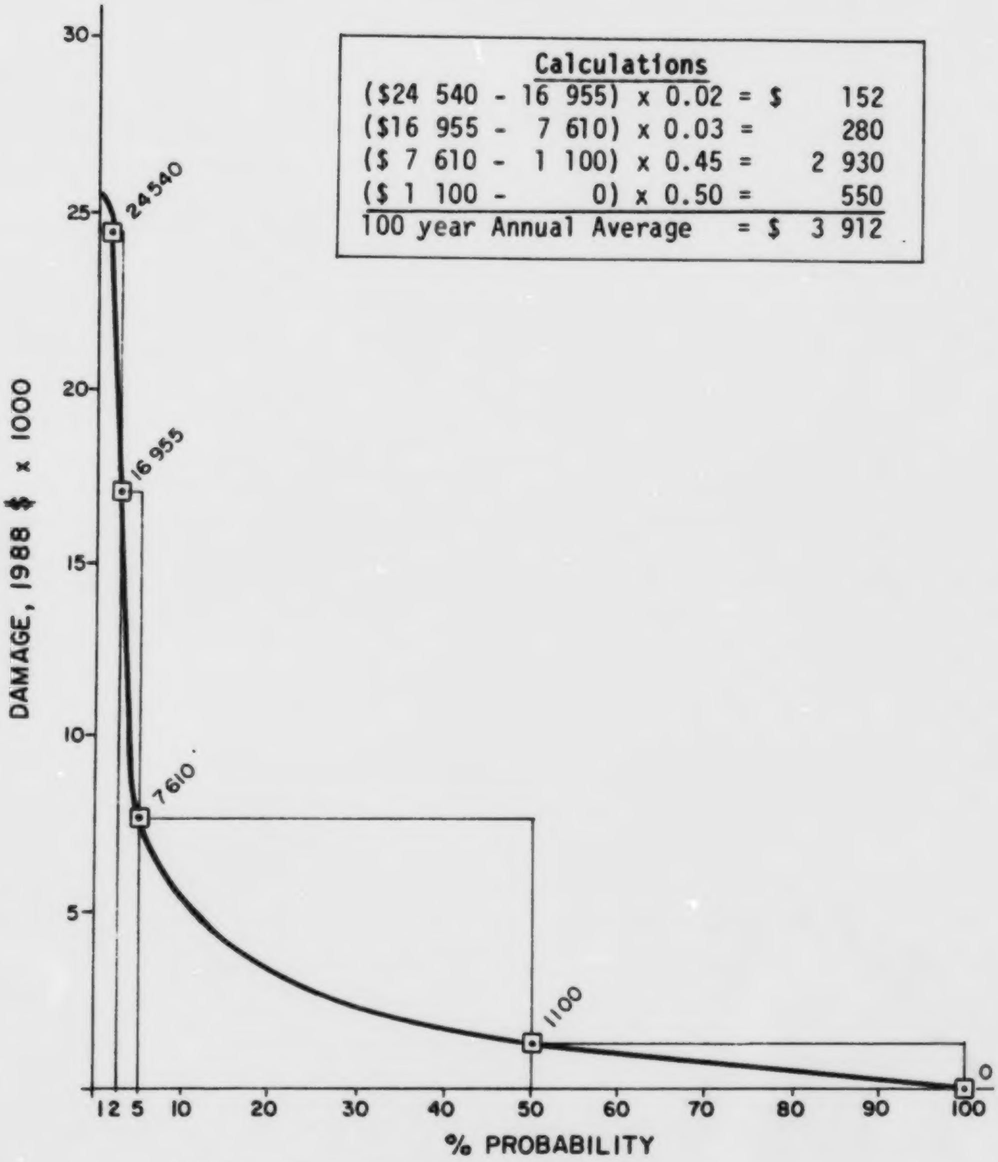
Calculations

($516 190 - 450 165) x 0.02 = $ 1 321
($450 165 - 253 405) x 0.03 = 5 903
($253 405 - 54 655) x 0.45 = 89 438
($ 54 655 - 0) x 0.50 = 27 328
100 year Annual Average = $123 990

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Campbellton Area
Estimated Average Annual Damage Curve and Calculations
(1988 $)

FENCO FIG. 5-3



Calculations

($24 540 - 16 955) x 0.02 = $ 152

($16 955 - 7 610) x 0.03 = 280

($ 7 610 - 1 100) x 0.45 = 2 930

($ 1 100 - 0) x 0.50 = 550

100 year Annual Average = $ 3 912

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Val D'Amour Community Area
Estimated Average Annual Damage Curve and Calculations
(1988 $)

FENCO FIG. 5-4

The expected average annual damages for the entire Campbellton Area are $123 990, while that for the community of Val D'Amour are $3 912, both in 1988$.

5.4.2 Present Value

The present value is calculated by summing the annual discounted values of each year of the discount period. The period is 50 years and a discount rate of 10% has been selected to conform with Federal Treasury Board recommendations. Sensitivity to rate changes will be provided by utilizing alternative discount rates of 5 and 15%. Table 5-12 provides the calculations at the 10% rate.

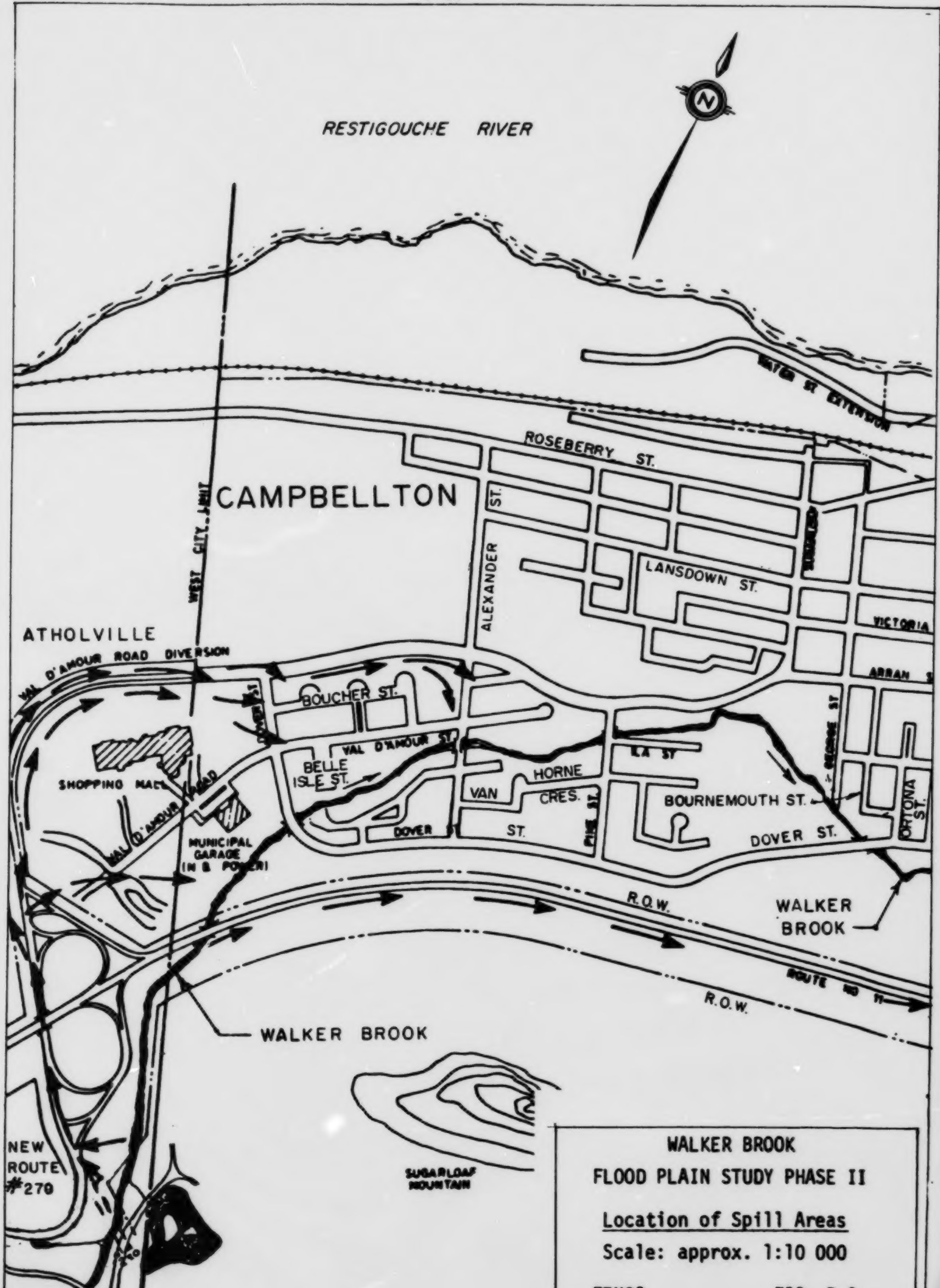
	Present Value	
Discount Rate	Campbellton Area	Val D'Amour Area
10%	$ 1 110 150	$ 35 000
15%	702 400	22 150
5%	2 174 500	68 550

TABLE 5-12
CALCULATION OF PRESENT VALUE AT 10% DISCOUNT RATE
(1988$)

Year	Campbellton Area	Val D'Amour Area
1	$111 591	$3 519
2	100 432	3 167
3	90 389	2 850
4	81 350	2 565
5	73 215	2 309
6	65 893	2 078
7	59 304	1 870
8	53 374	1 683
9	48 036	1 515
10	43 233	1 363
11	38 909	1 227
12	35 018	1 104
13	31 517	994
14	28 365	894
15	25 528	805
16	22 976	725
17	20 678	652
18	18 610	587
19	16 749	528
20	15 074	475
21	13 567	428
22	12 210	385
23	10 989	347
24	9 890	312
25	8 901	281
26	8 011	253
27	7 210	227
28	6 489	205
29	5 840	184
30	5 256	166
31	4 730	149
32	4 257	134
33	3 831	121
34	3 448	109
35	3 103	98
36	2 793	88
37	2 514	79

TABLE 5-12 (cont'd)
CALCULATION OF PRESENT VALUE AT 10% DISCOUNT RATE
(1988$)

Year	Campbellton Area	Val D'Amour Area
38	2 262	71
39	2 036	64
40	1 832	58
41	1 649	52
42	1 484	47
43	1 336	42
44	1 202	38
45	1 082	34
46	974	31
47	876	28
48	789	25
49	710	22
50	639	20
TOTAL	$1 110 151	$35 008



WALKER BROOK

FLOOD PLAIN STUDY PHASE II

Location of Spill Areas

Scale: approx. 1:10 000

FENCO FIG. 5-2

6.0 FLOOD DAMAGE REDUCTION OPTIONS

6.1 GENERAL

Several ways to reduce flood damages in Campbellton and Val D'Amour were presented in the Terms of Reference for this study and in earlier investigations (SMCC, 1985). These include flow diversion, dikes or flood berms, improvements to culverts and bridges, removal of flow obstructions, channel improvements, and other structural alternatives.

In addition to these approaches, there are also a series of non-structural measures such as contingency planning/early warning which can be effective for flood damage reduction. The first portion of this Chapter describes the structural measures (including flood proofing with berms) for flood damage reduction, and the last portion discusses non-structural measures.

6.2 STRUCTURAL MEASURES

The Flood Risk Map for the lower Walker Brook Area identifies the most sensitive areas for flooding as:

- the St. Albert area;
- Alexander Street to Pine Street;
- the lower crossing of Dover Street; and
- the Village Avenue crossing

Hence, the focus of the following assessment was first directed to these damage-prone areas.

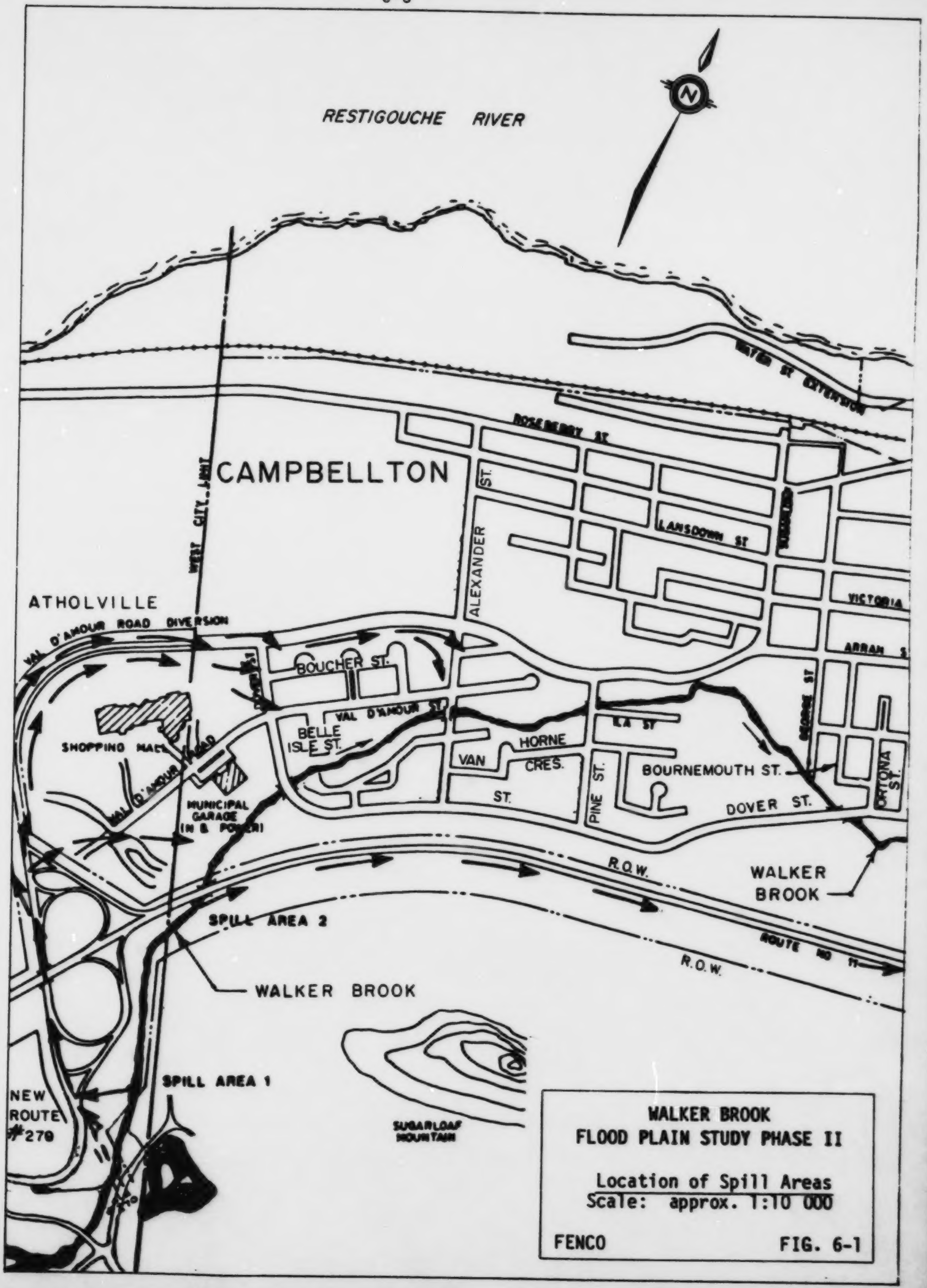
In general, a reduction in the peak flow reaching these areas will accomplish the goal of damage reduction. At St. Albert, the key is to eliminate out-of-channel spills which reach the area from the Route 11 and 270 interchange. At the other sites, there are advantages in redirecting flood flows around them, or improving the passage of flood flows through each area.

6.2.1 St. Albert Spill Area

Flooding in the St. Albert Area is cited earlier (SMCC, 1985) as arising principally as a result of spill from the main channel at the Route 11 interchange. Sedimentation and constriction at an old dam, as well as a channel constriction caused by the access ramp to the Route 11 overpass, are reported to create a spill condition. (See Figure 6-1, which follows).

Previous findings were based on three cross sections in the vicinity, and the spill capacity was estimated by considering it equal to the flow capacity of the western ditch of Route 270 at Route 11. Further reflection during this study suggests that:

- it would be advantageous to supplement previous physical survey data, topographic mapping and highway drawings with additional surveys to precisely define the limits of the spill area and the spill volumes;
- the spill location may be approximately 150 m upstream of the southernmost ramp to Route 11 at the sharp westward bend in Route 270, - perhaps extending over a length of about 120 m along the east side of this bend. This differs from the notion that the spill takes place north of the bend (Figure 6-2);
- "spill" in this area may be composed primarily of discharge in a western tributary (sub-watershed area #130 shown in Figure 3-6), which flows along the eastern ditch of the highway to join the main branch at the bend; and
- the "spill" may be greatly augmented by runoff from the hillsides which are west and north of St. Albert (western part of sub-watershed area #103, Figure 3-6). This area totals about 1.3 km^2.

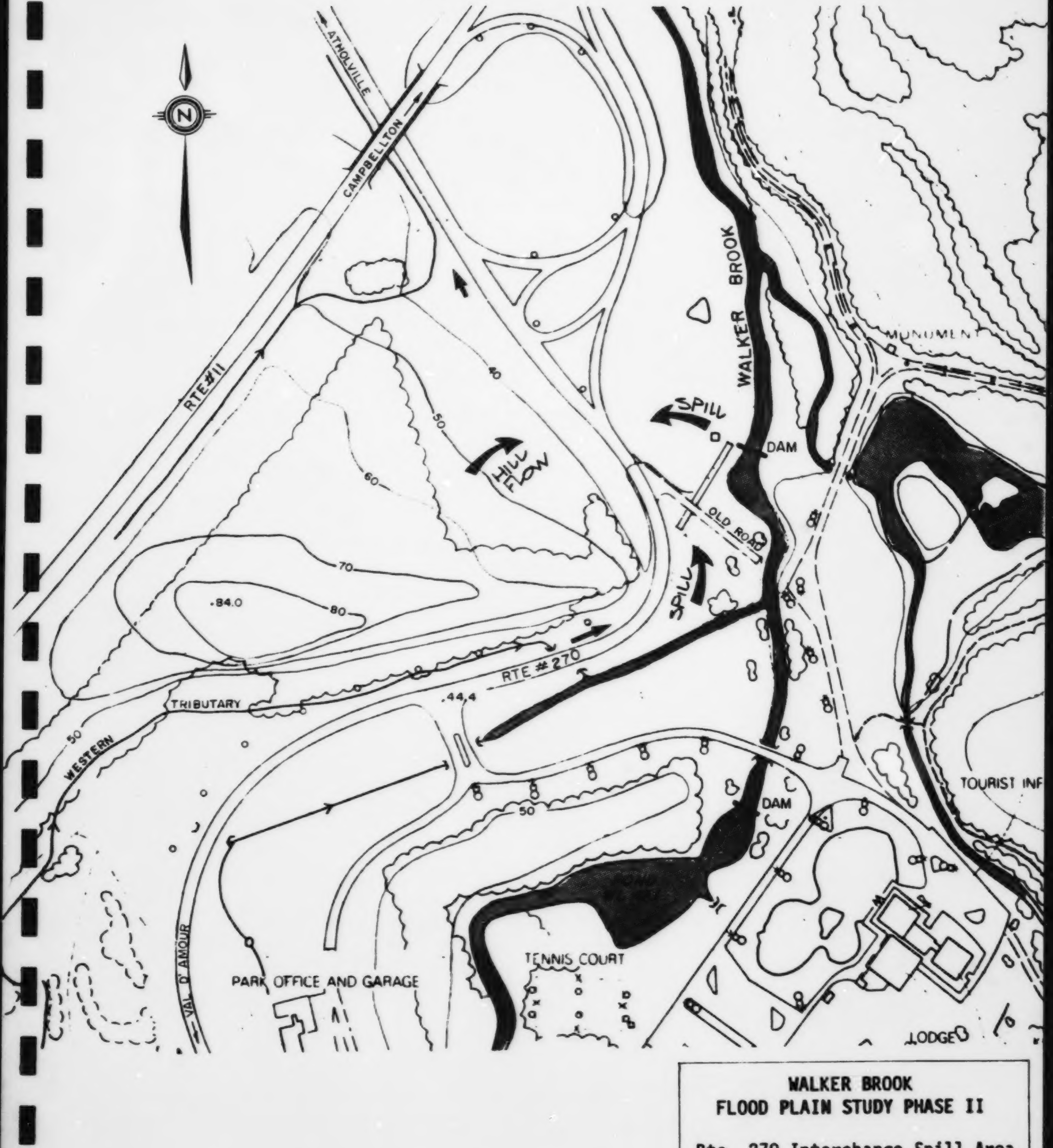


WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Location of Spill Areas
Scale: approx. 1:10 000

FENCO

FIG. 6-1



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Rte. 270 Interchange Spill Area
Scale: approx. 1:5 000

FENCO FIG. 6-2

In all, there are several possible contributors to spill flow into the St. Albert area. Although their volumes cannot be precisely defined from field surveys, they are estimated in the following sections which describe the flood damage reduction options advanced for benefit-cost analysis.

Options

(i) Spill from the main channel

Spill at this location appears controllable by the following options (Figure 6-3):

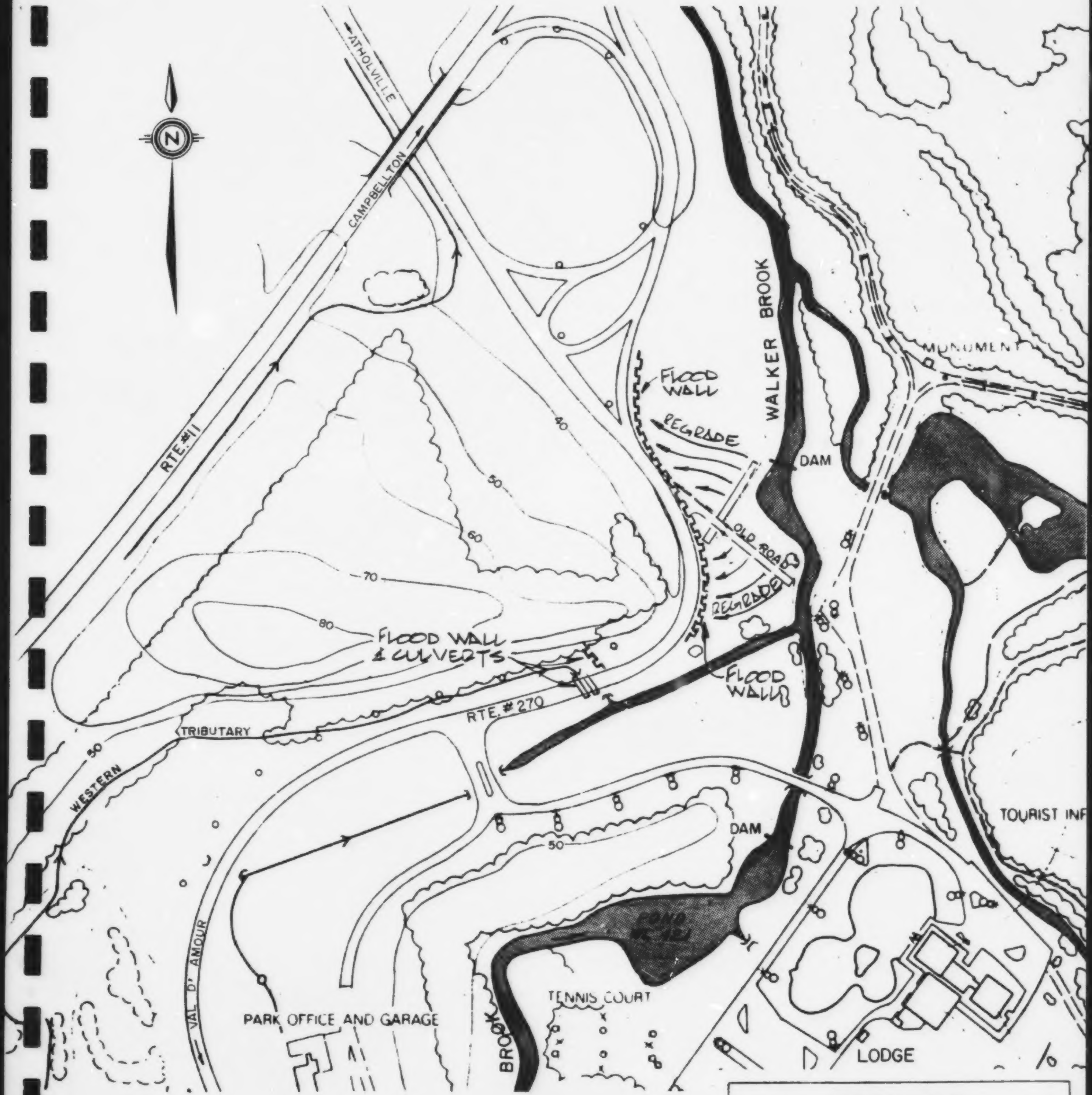
a) regrading former roadbed material, which now forms an "L"-shaped flow constriction in the flood plain. The upper limb of the "L" may be graded into a low berm along the north-south portion of the highway at the bend, and the lower limb of the "L" may do the same at the bend.

   The desired elevation of the top of this berm is estimated to be approximately 0.5 m above the existing grade of the highway on the east side.

b) construction of a low floodwall along the east side of the highway at the bend. This wall (approximately 120 m long, 1.0 m high at most) would be incorporated with a guard rail.

c) a combination of regrading with a floodwall. The regrading is desirable to open up the floodway and reduce flood levels. The floodwall (perhaps 0.5 m high, excluding freeboard) would provide additional protection along the east side of the roadway.

(ii) "Spill" from the western tributary, sub-watershed area #130



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Options
@ Rte. 270 Interchange Area
Scale: approx. 1:5 000

FENCO

FIG. 6-3

This tributary crosses the highway via a culvert just west of the bend in Route 270 (about 90 m east of the entrance to the Sugarloaf Park complex). Design flows for this branch are estimated from hydrologic modelling in Chapter 3.0 to be:

Return Period	1:2 Years	1:20 Years	1:50 Year	1:100 Years
Flow (m^3/s)	2.7	5.4	6.5	7.3

These flow estimates are based on ratios of AMC-3 rainfall events (Table 3-9) and design flow values (Table 3-11).

It cannot presently be ascertained whether or not this culvert becomes blocked or iced up, but it would be appropriate to ensure culvert capacity at this location is sufficient to pass the above-listed high flows. This would entail culvert expansion and construction of a flood wall (typically to 0.5 m above existing road level) to stop flows from bypassing the culvert and carrying downstream to St. Albert.

(iii) "Spill" from the area west of the drive-in theatre

Runoff from this area (1.3 km^2) is directed to the ditches along the west side of Route 270/north side of the Val D'Amour Road Diversion, to ultimately join the main stream near Pine Street.

It is considered herein that ditch, sewer and culvert capacities are sufficient in this portion of Atholville/Campbellton to carry these inflows without flooding, once the excess flows from upstream (western tributary and spill from the main channel) are contained in the main channel through Options (i) and (ii) discussed above.

6.2.2 Spill Area Upstream of Culvert Crossing at Route 11 (Diversion Along Route 11)

Flow diversions along the southern drainage ditch of Route 11 are reported to be frequent, with the diverted flow passing through a culvert (or overtopping Route 11) to rejoin the main stream near Gallant Drive (spill area 2 shown in Figure 6-1). Previous study (SMCC, 1985) determined that the downstream culvert has a capacity of about 15 m^3/s before Route 11 is overtopped.

The previous surveys and hydrotechnical study of the Route 11 culvert and diversions identified that the existing "natural" diversion at the highway crossing partitions flows approximately as follows (SMCC, 1985):

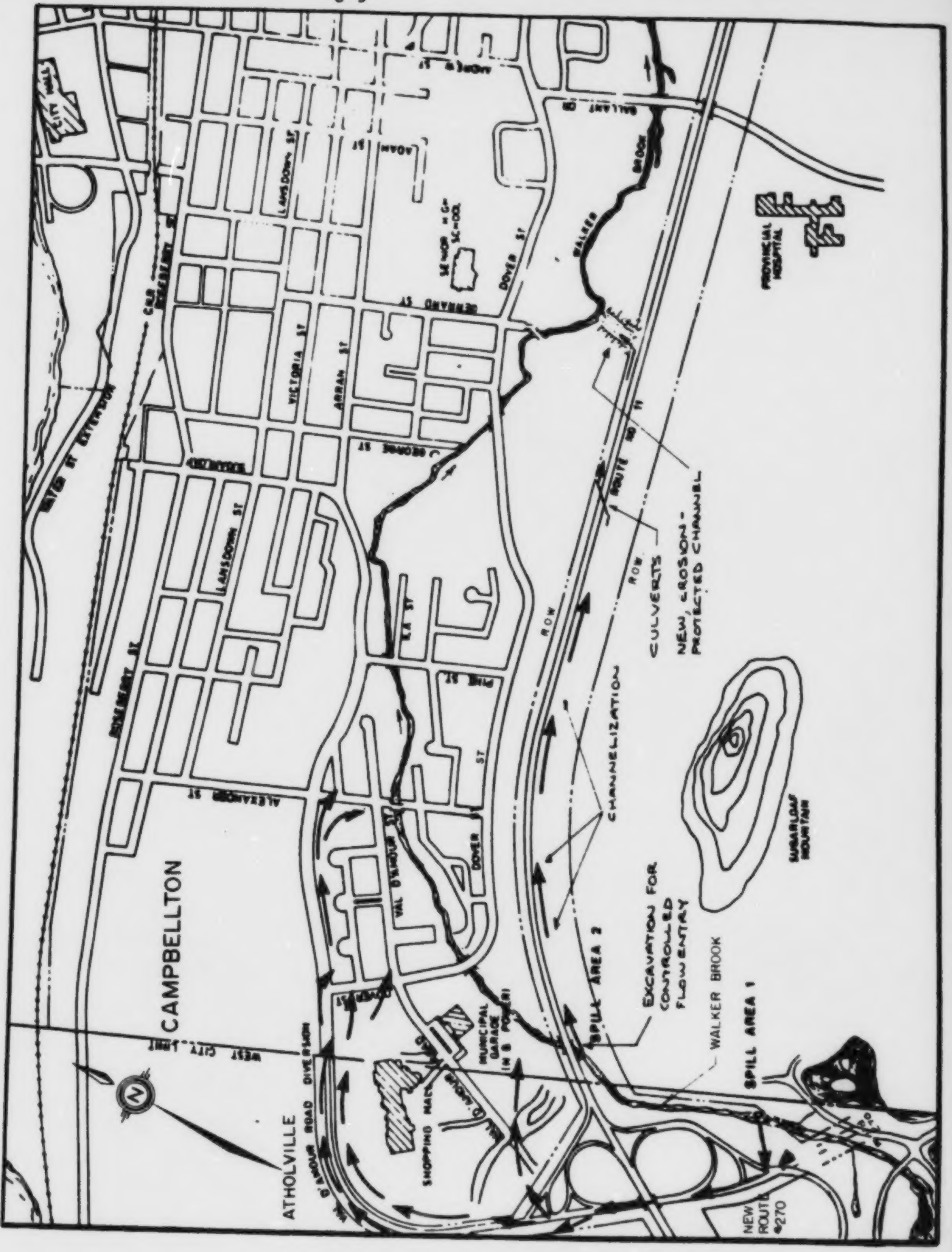
Return Period (Years)	Flow in Main Stream + (m^3/s)	Flow in Highway Ditch = (m^3/s)	Total (m^3/s)
1:2	19.8	4.0	23.8
1:20	24.0	22.2	46.2
1:50	25.0	30.5	55.5
1:100	26.5	36.2	62.7

This natural diversion could be augmented by a weir at the culvert or by additional excavation. For the present, the Route 11 channel (to give the above flow partitioning) was taken to be a trapezoidal shape with a bottom width of 20 m. It has been sized to carry 40 m^3/s.

A permanent flood diversion channel in this area would be feasible (Figure 6-4) provided that it included:

- excavation at the upstream Route 11 culvert to allow controlled entry of flows into the diversion "channel" (an area where existing topographical data is too limited for any detailed assessment). It currently appears that this should not be a significant expenditure.

- channelization of the Route 11 ditch to ensure sufficient



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FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Options Along Rte. 11
Scale: approx. 1:12 000

FENCO FIG. 6-4

capacity to keep depths and velocities within codes of practice for highway design.

- a larger or replacement culvert (or several additional culverts) across the highway upstream of Gallant Drive to carry the diversion flow back to the river. An appropriate location for these culverts is about 500 m west of Gallant Drive.

- channelization and channel protection (e.g. riprap) for the channel which reconnects the diversion flow to the main stream upstream of Gallant Drive.

6.2.3 Bridge Expansions

The channel constriction at Gallant Drive was removed in 1987 by the construction of a new open span bridge. It is similarly proposed to improve the channel with a new bridge at Village Avenue (Figure 6-5). In both cases, which we consider to be the "existing" situation, the increased flow capacity has reduced flood levels.

Table 6-1, which follows, summarizes the effect of the proposed new bridge construction at Village Avenue at several representative cross sections upstream of the bridge. Water levels are substantially reduced at the bridge and upstream at Gallant Drive.

Table 6-2 gives the effect of the new bridge at Gallant Drive. The street is no longer inundated by flood waters and upstream levels are significantly reduced over a channel length of about 350 m.

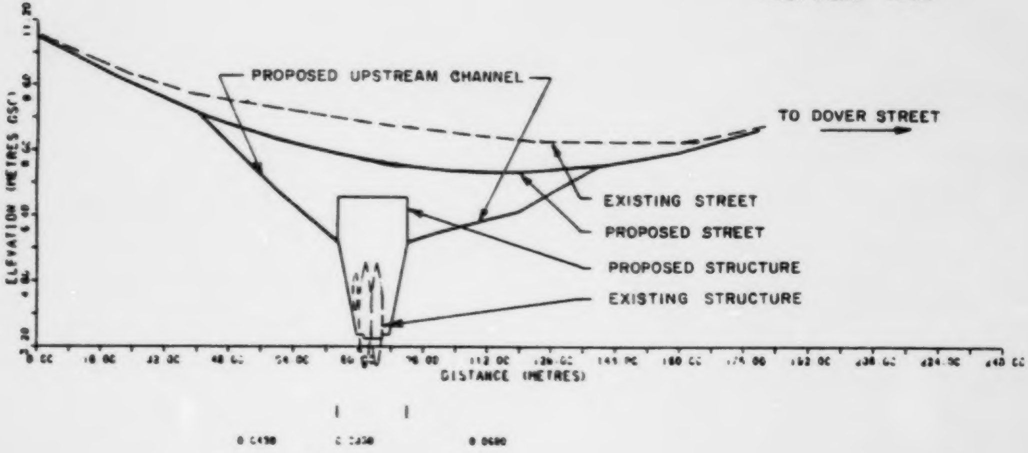
It was anticipated that similar reductions in flood levels could be achieved at other locations and replacements for the following culverts were also examined herein:

Dover Street (lower crossing)

WALKER BROOK - 1:100 Y HEC-2 MODEL
CROSS SECTION NUMBER 37.00

VILLAGE AVENUE CROSSING

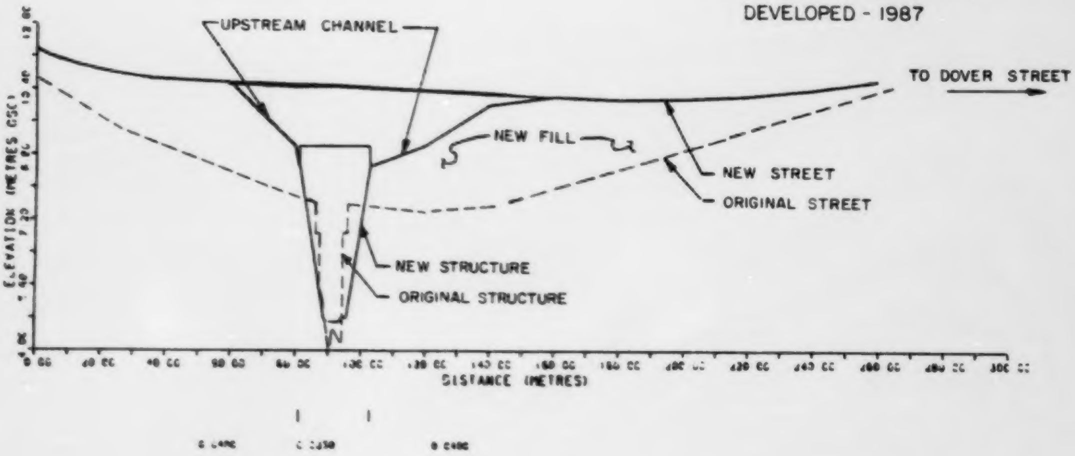
WALKER BROOK BRIDGE No 3
PROPOSED - 1988



WALKER BROOK - 1:100 Y HEC-2 MODEL
CROSS SECTION NUMBER 36.

GALLANT DRIVE CROSSING

WALKER BROOK BRIDGE No.4
DEVELOPED - 1987



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Options:
Gallant Drive &
Village Ave. Crossings

FENCO FIG. 6-5

TABLE 6-1

VILLAGE AVENUE - EFFECT OF PROPOSED NEW BRIDGE

Location	Section No.		Return Period Level (m) 1:20 Year	Return Period Level (m) 1:100 Year
Bridge	37.0	New*	5.37	5.68
		Old**	8.19	8.19
Upstream	37.1	New*	5.78	6.12
		Old**	8.65	8.99
Near Gallant	536.8	New*	6.95	6.99
		Old**	8.66	9.00

TABLE 6-2

GALLANT DRIVE - EFFECT OF EXISTING NEW BRIDGE

Location	Section No.		Return Period Level (m) 1:20 Year	Return Period Level (m) 1:100 Year
Bridge	36.0	New*	7.02	7.16
		Old**	8.72	9.06
Upstream	36.1	New*	7.71	7.98
		Old**	8.73	9.07
Near Tingley Cr.	36.3	New*	8.71	8.87
		Old**	8.89	9.17
Just east of School	535.0	New*	9.51	9.75
		Old**	9.59	9.98

No further reduction in levels upstream of school.

* flood levels with "new" bridge/structure and updated flood flows

** flood levels with "old" bridge/culvert (from SMCC, 1985)

Pine Street
Dover Street (upper crossing)

6.2.3.1 Dover Street (Lower Crossing). The roadway at this location slopes downward from the southern bank to a plateau of lower elevations between Ortorna and Gerrard Streets. Flood flows exceed the culvert capacity causing an upstream backwater and spill over the road.

Several trials were undertaken to determine the reduction in flood levels achievable through:

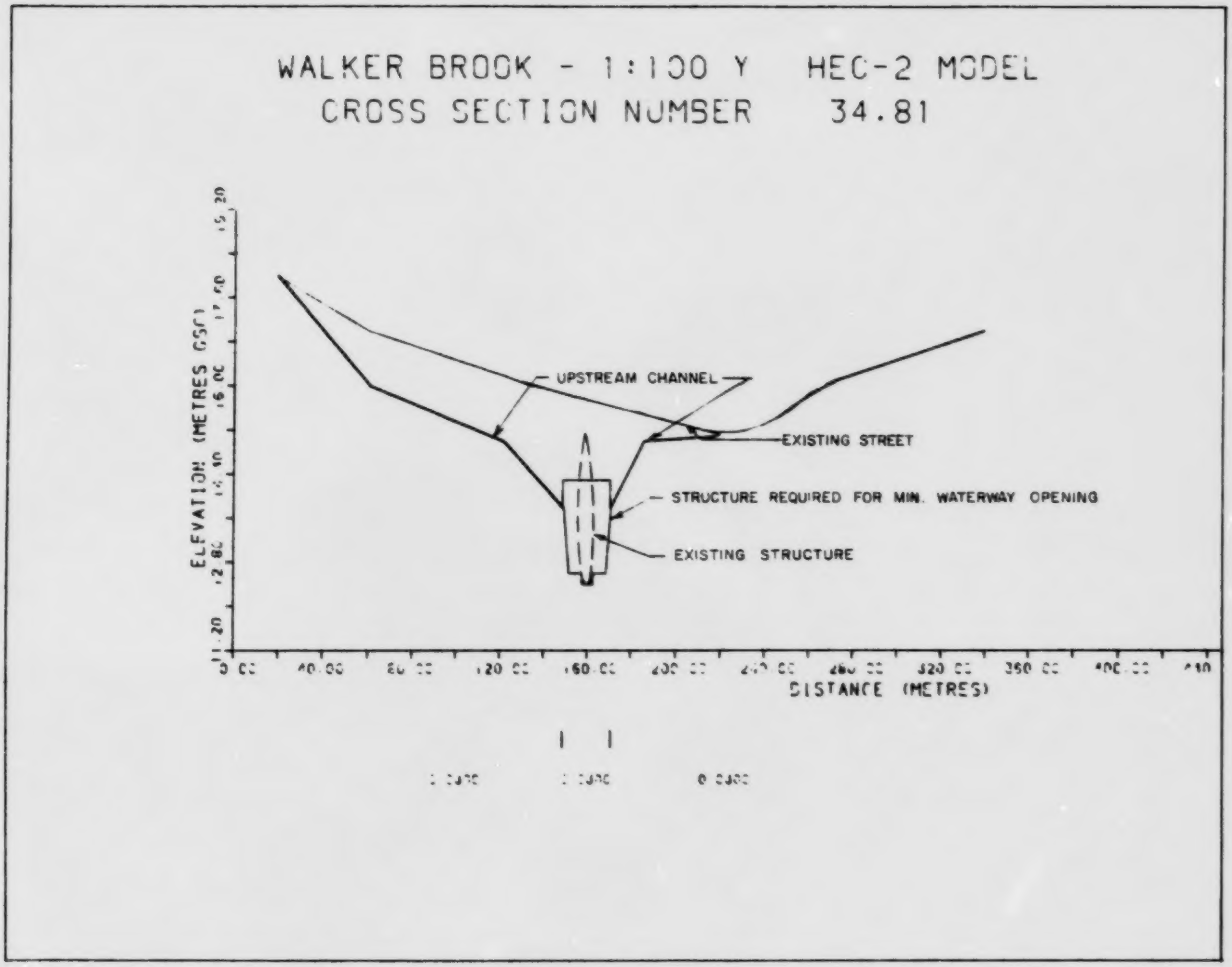
- the addition of a second culvert (total conveyance area 22 m^2);

- culvert replacement by a new bridge (similar to Gallant Drive, conveyance area 33 m^2);

and combinations of culverts with diking and channelization. The bridge (sketched in Figure 6-6) proved effective in eliminating spill over the road and was advanced for costing purposes.

Table 6-3 summarizes the reduction in flood levels from such a bridge. There is a substantial improvement at the bridge, and general lowering of water level for a distance of 350 m upstream. However, the flood plain is flat and the improvements at the bridge do not make a major difference in the areal extent of the flood plain.

6.2.3.2 Pine Street. Pine Street is relatively flat from John Street to Val D'Amour Road Diversion, but lowest at its intersection with the latter. The existing culvert is undersized for high flood flows and, hence, the roadway is prone to flooding from Ila Street to Val D'Amour Road Diversion. The backwater for the road crossing also contributes to flooding along Van Horne Crescent and the Val D'Amour Road Diversion at Pine Street.



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Options
Dover Street (Lower Crossing)

FENCO **FIG. 6-6**

TABLE 6-3

DOVER STREET (LOWER CROSSING) - EFFECT OF A NEW BRIDGE

Location	Section No.		Return Period Level (m) 1:20 Year	1:100 Year
Bridge	34.81	Current*	15.64	15.69
		Previous**	16.99	16.99
		(Revised)***	(14.04)	(14.39)
Upstream	33.20	Current	15.83	16.04
		Previous	17.03	17.06
		(Revised)	(15.23)	(15.36)
Sugarloaf	533.0	Current	16.68	17.02
		Previous	17.04	17.07

No further reduction in levels upstream of Section 533.0.

* flood levels with "current" flood flow estimates
** with previous flood flow estimates (SMCC, 1985)
*** flood levels with revised, new bridge crossing

Several options were examined for improving the crossing and the selected bridge case is plotted in Figure 6-7. This opening is sufficient to pass the 1:100 year flood and its effect on reducing upstream levels is given in Table 6-4. As at Dover Street, the benefit derived from opening the bridge extends upstream, in this case about 325 m upstream to Alexander Street.

6.2.3.3 Dover Street (Upper Crossing). The current Dover Street culvert has a slightly smaller opening than at Pine Street. This causes a flow constriction and backwater flooding around the NB Power property, and would result in spill over Dover Street (south of Val D'Amour Street) during 1:100 year flood conditions.

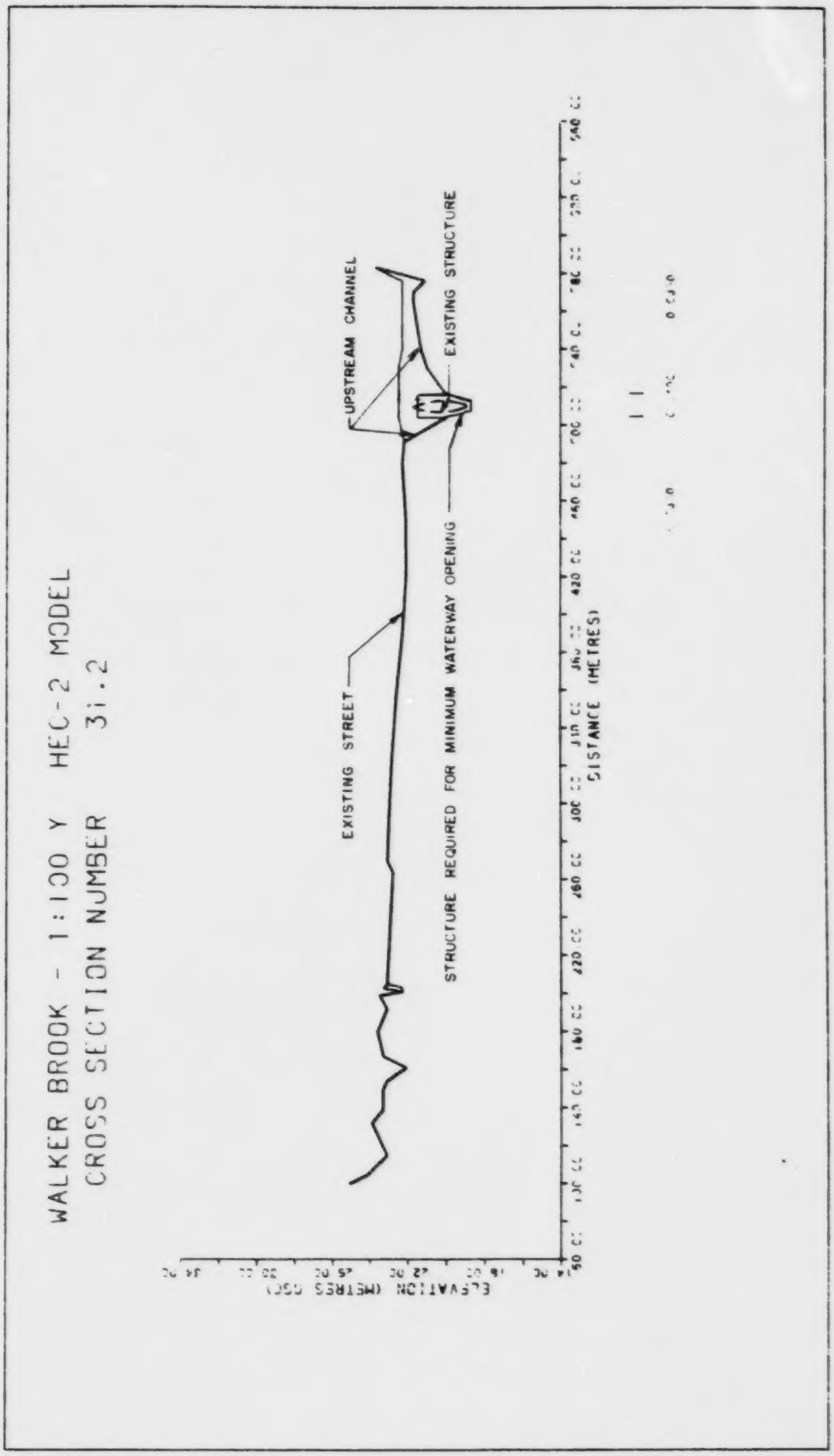
An appropriate replacement bridge for the existing culvert is shown in Figure 6-8, and Table 6-5 provides examples of the reduction in flood levels provided by such a bridge. This bridge would pass the 1:100 year flood flow without overtopping Dover Street and without flooding at the NB Power property.

6.2.4 Flow Obstructions

(i) Lower reach

The principal flow obstruction in the lower portion of the study area is the abandoned dam between Village Avenue and the mouth of Walker Brook. The configuration of the existing ruins is shown in Figure 6-9, along with a bottom profile which could be achieved by removal of the ruins.

The existing condition and water levels with removal of the ruins are compared in Table 6-6. The table illustrates that water levels can be lowered by removing the flow obstruction, but that this removal only affects levels between Village Avenue and the old dam. Noteworthy is that the reduced levels do not substantially decrease the areal extent of the flood risk area, and that there are no damage-prone structures in this reach.



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Options
Pine Street

FENCO **FIG. 6-7**

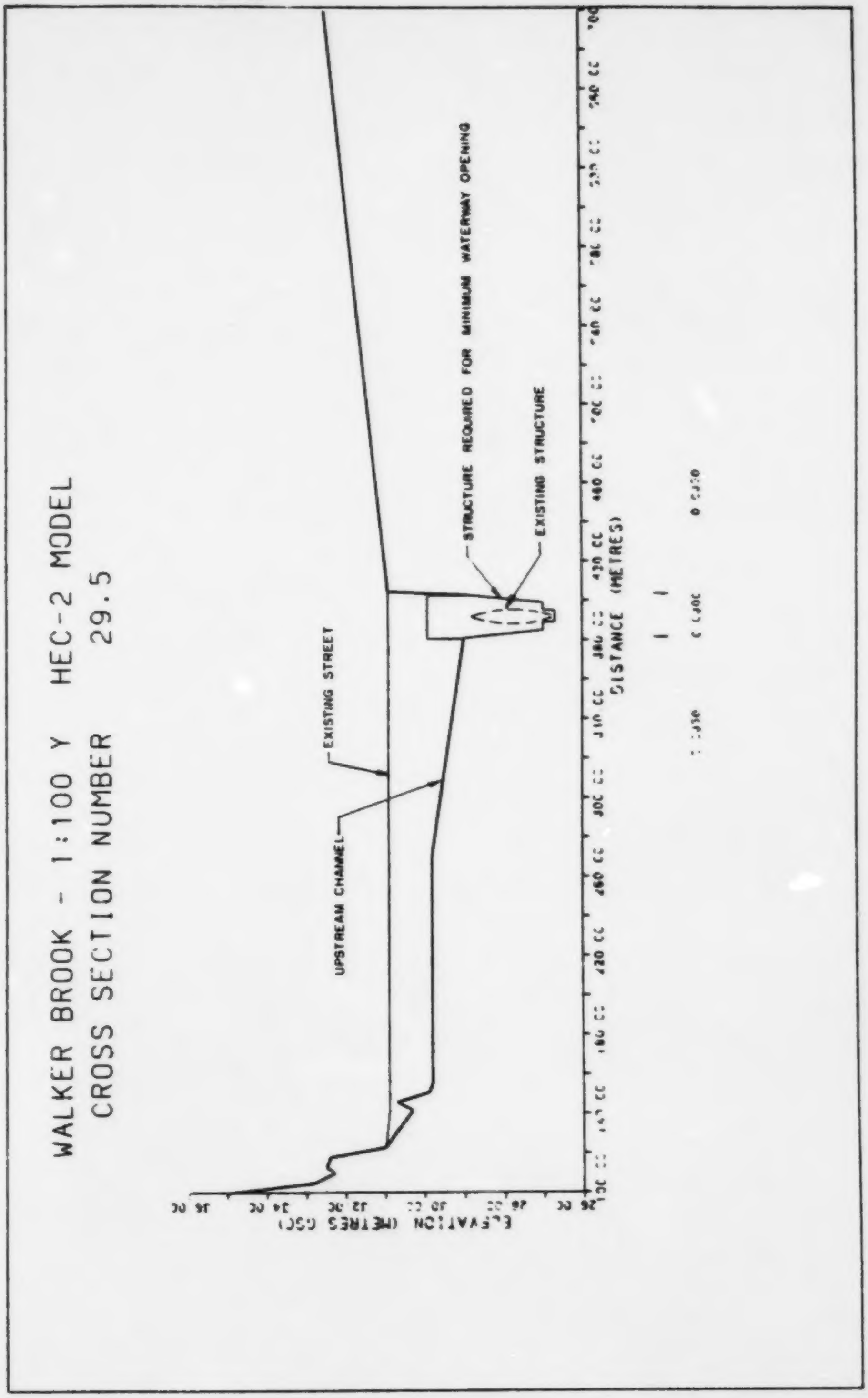
TABLE 6-4

PINE STREET - EFFECT OF A NEW BRIDGE

Location	Section No.		Return Period Level (m) 1:20 Year	1:100 Year
Bridge	31.2	Current*	22.31	22.46
		Previous**	22.40	22.51
		(Revised)***	(20.70)	(21.00)
Upstream	30.1	Current	22.34	22.49
		Previous	22.41	22.53
		(Revised)	(21.69)	(21.96)
Van Horne Cr.	30.25	Current	22.51	22.70
		Previous	23.60	24.07
		(Revised)	(22.41)	(22.61)
at Alexander	530.8	Current	23.83	24.54
		Previous	24.21	24.89
		(Revised)	(23.82)	(24.56)

Effect of new bridge limited to area between Pine and Alexander Streets.

* flood levels with "current" flood flow estimates
** with previous flood flow estimates (SMCC, 1985)
*** flood levels with revised, new bridge crossing



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option:
Dover Street (Upper Crossing)

FENCO FIG. 6-8

TABLE 6-5

DOVER STREET (UPPER CROSSING) - EFFECT OF A NEW BRIDGE

Location	Section No.		Return Period Level (m) 1:20 Year	 1:100 Year
Bridge	29.50	Current*	29.82	30.11
		Previous**	29.55	29.70
		(Revised)***	(28.39)	(28.65)
NBEPC	28.00	Current	30.18	30.18
		Previous	29.86	29.95
		(Revised)	(28.76)	(29.33)
Near Rte. 11	528.00	Current	30.13	30.08
		Previous	30.24	30.64
		(Revised)	(29.94)	(30.02)

No reduction in flood levels achieved above Route 11 crossing.

TABLE 6-6

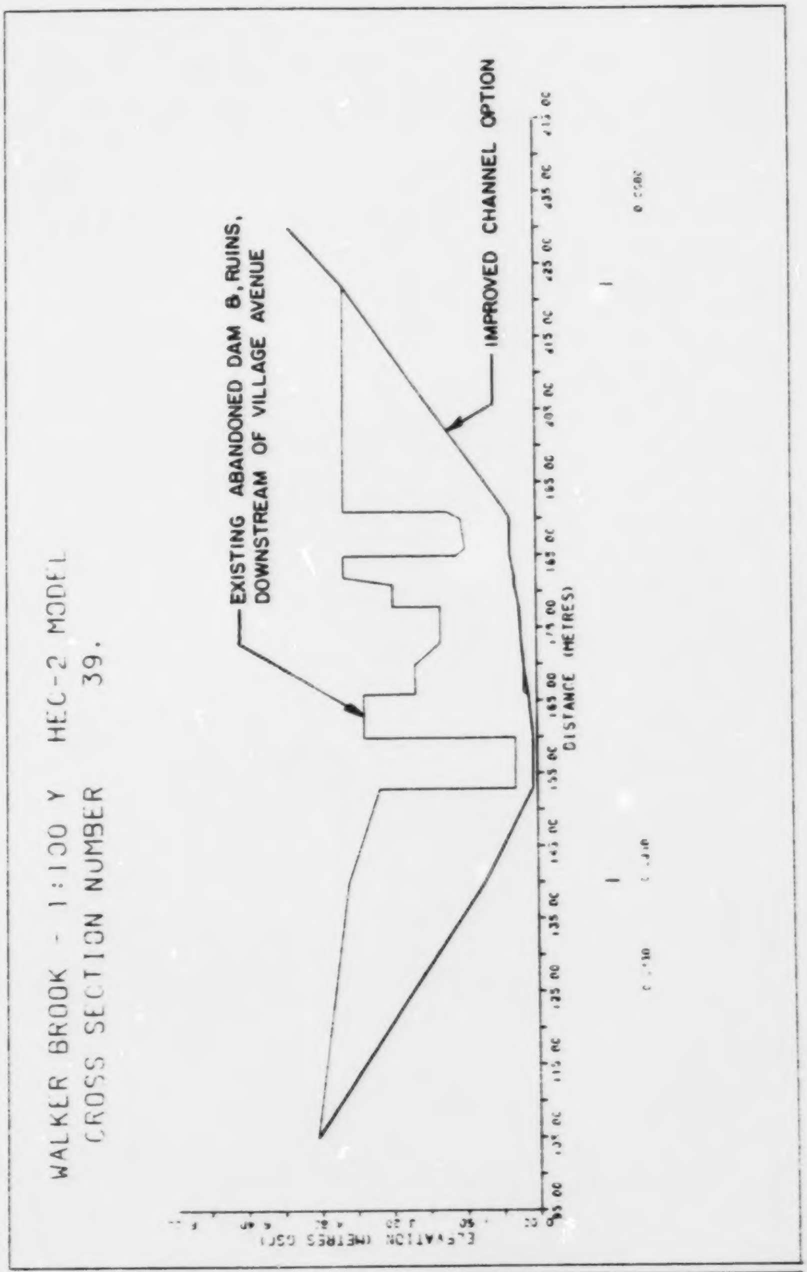
SITE OF DAM RUINS - EXISTING AND EFFECT OF OBSTRUCTION REMOVAL

Location	Section No.		Return Period Level (m) 1:20 Year	 1:100 Year
Dam	39.0	Current*	2.81	3.13
		Previous**	2.91	3.32
		(Revised)***	(1.82)	(2.15)
Upstream	538.0	Current	2.87	3.04
		Previous	3.19	3.49
		(Revised)	(2.87)	(2.97)
Village Ave.	537.8	Current	4.88	5.15
		Previous	6.15	6.68
		(Revised)	(4.89)	(5.16)

* existing conditions with "current" brook crossing and flood flows

** flood levels with previous flood flow estimates (SMCC, 1985)

*** flood levels with revised, new bridge crossing (or obstruction removal)



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option: Abandoned Dam Removal

FENCO FIG. 6-9

The weir/dam below Sugarloaf Park was previously (SMCC, 1985) considered as the obstruction contributing to spills at Route 270. This weir does not now appear to be as significant as the nearby, upstream constriction in the channel overbanks, which was discussed in Option (i) of Section 6.2.1. Hence, removal of this weir would not appear to be warranted.

(ii) Existing Route 11 Crossing

The existing 5 050 mm (S) x 3 300 mm (R) pipe-arch culvert, with a bevelled end inlet, has a capacity of only approximately 60% of the estimated 1:100 year flood flow at this point on the system, even if there were not the current natural tendency for floodwaters to divert to the existing south ditch of Route 11 once they reach a certain level.

The crossing capacity could be remedied by the installation of an additional pipe-arch culvert, 3 890 mm (S) x 2 690 mm (R), to take these flood flows. In conjunction with this installation, and to best utilize the available capacity of both pipes, the construction of headwalls would be required, together with immediate upstream channelization, erosion protection works, and a cut-off dike across the existing south ditch of Route 11.

6.2.5 Diking and Channelization

Diking to contain flood flows within the channel was examined at several locations.

6.2.5.1 Between Route 11 and Dover Street (Upper Crossing). This option is suggested to eliminate: flooding at the NB Power building; spill into storage areas south of it; possible spills near the Honda dealership; and, uncontrolled spills over Dover Street. To accomplish this would require approximately:

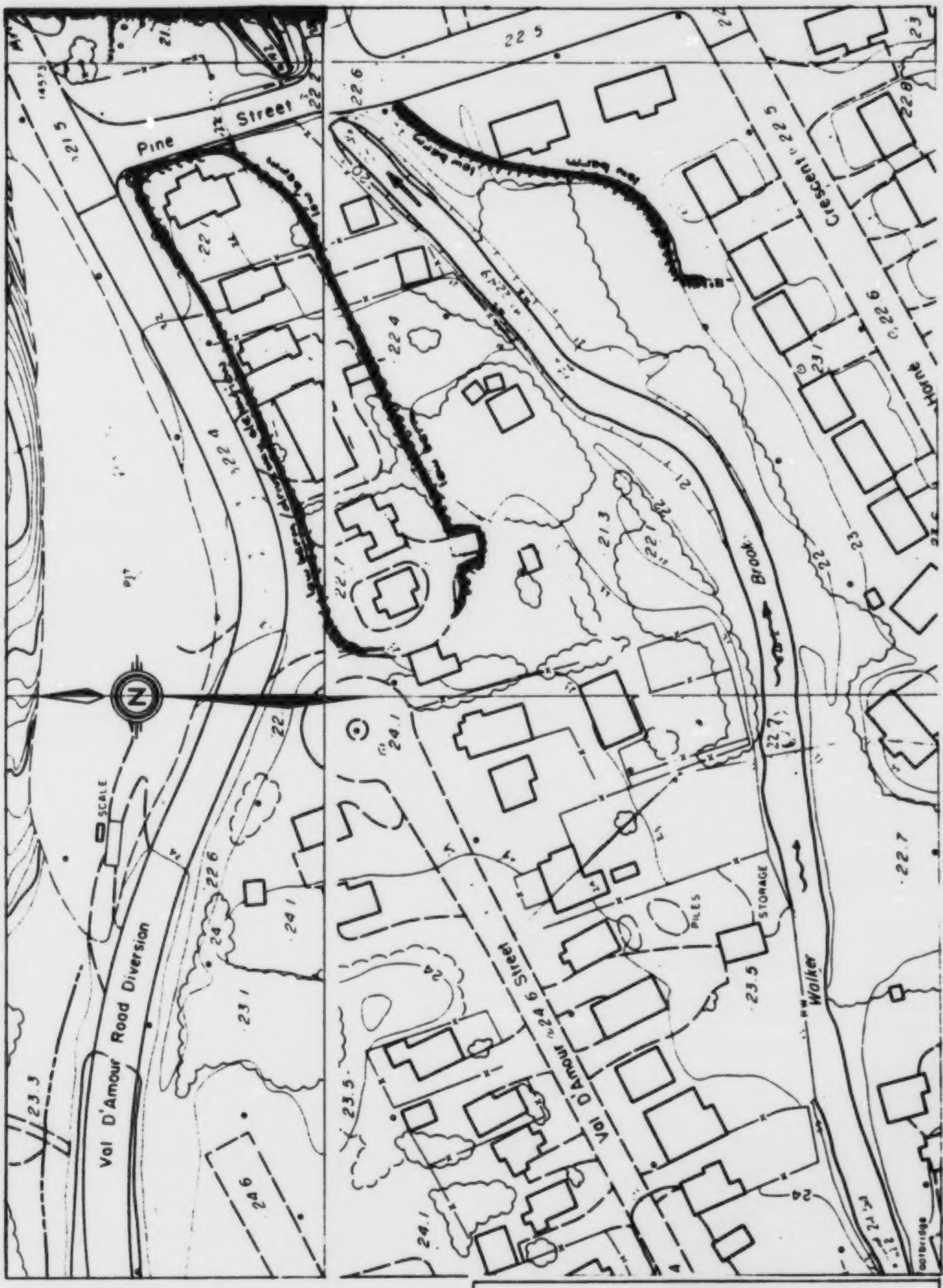
- 275 m of diking along the northwest bank from Route 11 to Dover Street;
- 75 m of diking along the southeast bank near the Honda dealer; and
- raising of the grade at Dover Street.

In essence, this option transforms the brook channel into a reservoir which will reach a water level sufficiently high to force all the flood flow through the existing culvert (9.2 m^2 area). This concept fails because the culvert is too small to carry the flow and dikes would have to be excessively high (about six metres in height).

6.2.5.2 Above Pine Street. Dikes along both banks of the river at this location could be used to eliminate flood damages to structures along Pine Street and at its intersection with the Val D'Amour Road Diversion (Figure 6-10). This option requires approximately:

- 100 m of low berm (about 1 m maximum height) along the southern bank near Van Horne Crescent; and
- a low circular berm (about 300 m in length, 0.5 m high) surrounding the six buildings/homes at the southwest corner of Val D'Amour Road Diversion and Pine Street. Some of the entrance driveways from Val D'Amour Road Diversion would need to be elevated to effect this protection and internal drainage provisions would be required.

This option protects the main structures, but still results in spill over Pine Street and flooding of the Pine/Val D'Amour Road Diversion intersection. It does not substantially increase or decrease water levels (Table 6-7). Buildings in this area may also be flood proofed by elevating them, which is discussed later in Section 6.3.2.



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option:
Diking Above Pine Street
Scale: approx. 1:1600

FENCO FIG. 6-10

TABLE 6-7

PINE STREET - DIKING ALTERNATIVE

Location	Section No.		Return Period Level (m)*** 1:20 Year	1:100 Year
Bridge	31.2	Current*	22.31	22.46
		Previous**	22.40	22.51
		(Diked)***	-	(22.38)
Upstream	30.1	Current	22.34	22.49
		Previous	22.41	22.53
		(Diked)	-	(22.49)
Van Horne	30.25	Current	22.51	22.70
		Previous	22.60	24.07
		(Diked)	-	(22.69)
at Alexander	530.8	Current	23.83	24.54
		Previous	24.21	24.89
		(Diked)	-	-

* existing conditions with "current" brook crossing and flood flows

** flood levels with previous flood flow estimates (SMCC, 1985)

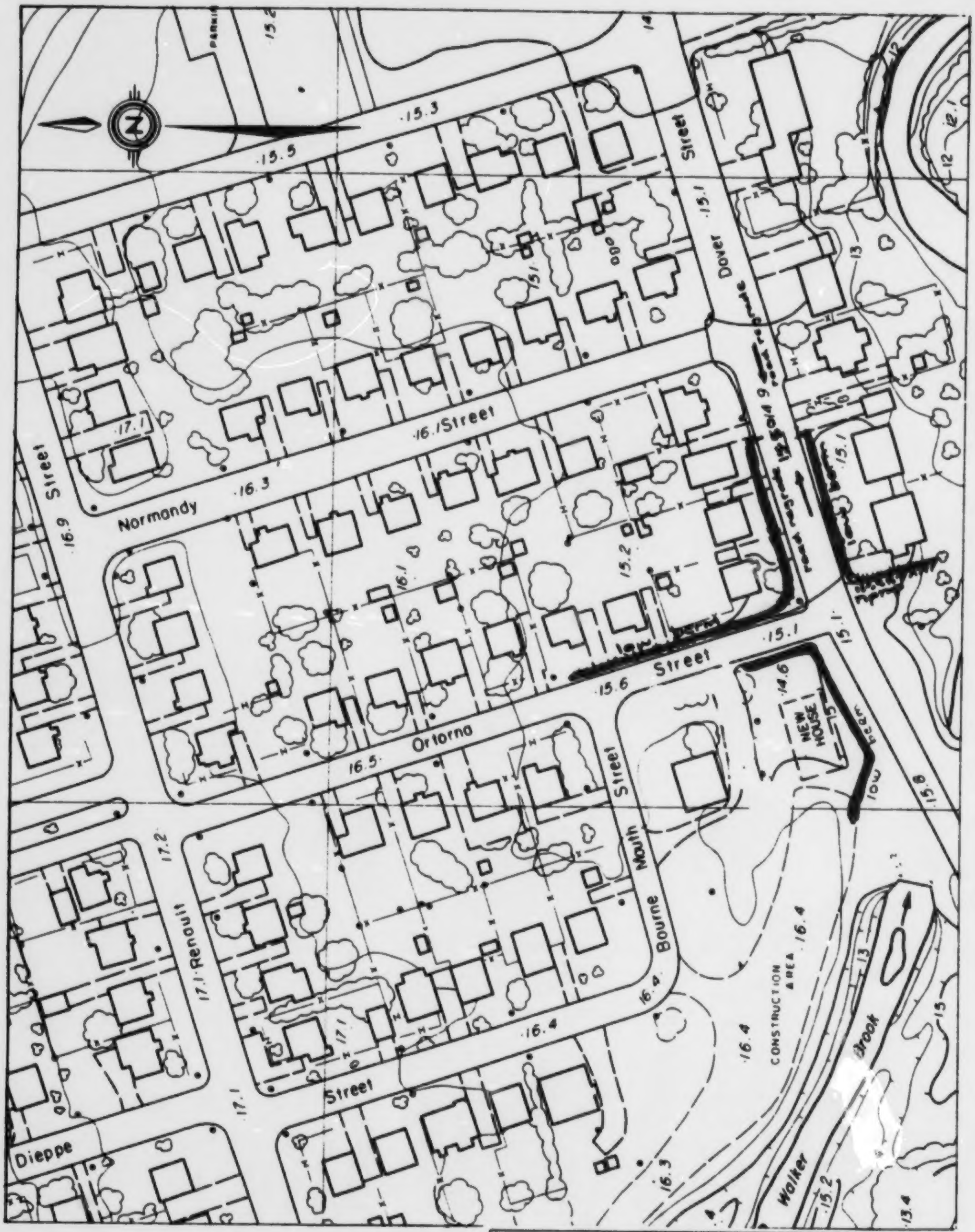
*** flood levels with revised (expanded) cross section or with diked banks

6.2.5.3 At Dover Street (Lower Crossing). The flood profile of this location is dictated by the profile of Dover Street, which is lowest on the north bank near Ortorna Street. Flood flows are projected to spill over the north bank and eastward across Ortorna and along Dover to Gerrard, and flow back into the river between the bridge and Gerrard Street.

Dike protection here is best achieved by low berms (Figure 6-11):

- at the southwest corner of Bourne Mouth and Ortorna Streets;
- at the northwest corner of Ortorna and Dover Streets;
- at the southeast corner of Ortorna and Dover Streets (riprap protection would be required here);

and by regrading Dover Street to a high spot level of 15.5 m between Ortorna and Normandy Streets. This option does not lower flood levels and there would still be frequent occasions when Dover Street and Ortorna Street would be flooded by spills.



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option:
Diking at Dover Street
(Lower Crossing)
Scale: approx. 1:1600

FENCO FIG. 6-11

6.2.5.4 Channelization - Alexander Street. The cross section at the Alexander Street footbridge presents a narrow flow contraction which elevates upstream water levels in part of St. Albert. Expansion of this section, simply by regrading the banks would smooth passage through the area (Table 6-8). An additional span would have to be added to the existing footbridge, or it would have to be replaced by a longer span to implement this improvement.

6.2.5.5 Channelization - Dover Street to Gallant Drive. The new Gallant Drive bridge lowers flood levels in this area (Table 6-2). There is now no flood damage to structures, and no damage reduction benefit to be derived by channel improvements. However, the area of flood-prone land could be reduced by a channel such as that sketched in Figure 6-12. As noted on Figure 6-12, low dikes or protected fill material would be required on both sides of the channel in the lower portion near Gallant Drive.

6.2.5.6 Channelization - Downstream of Village Avenue. In addition to removing the ruins of the old dam in this reach (Figure 6-9), the size of the flood risk area could be reduced by a channelization design such as shown in Figures 6-13. This design includes moderate diking or protected fill along both sides of the channel. As above, no reduction in flood damages would be achieved by this activity because no structures are within the flood risk area.

6.2.6 Val D'Amour Area

The principal cause of overbank flooding in Val D'Amour in the past appears to have been channel constrictions caused by undersized culvert crossings, and undoubtedly localized bank erosions. In recent years, some crossings have been washed out and replaced, while others have been abandoned after being destroyed by flood flows. In all, a full appreciation of the magnitude of flood flows appears to have been underestimated when these crossings were originally developed.

TABLE 6-8

ALEXANDER STREET - EFFECT OF A WIDER OPENING AT FOOTBRIDGE CROSSING

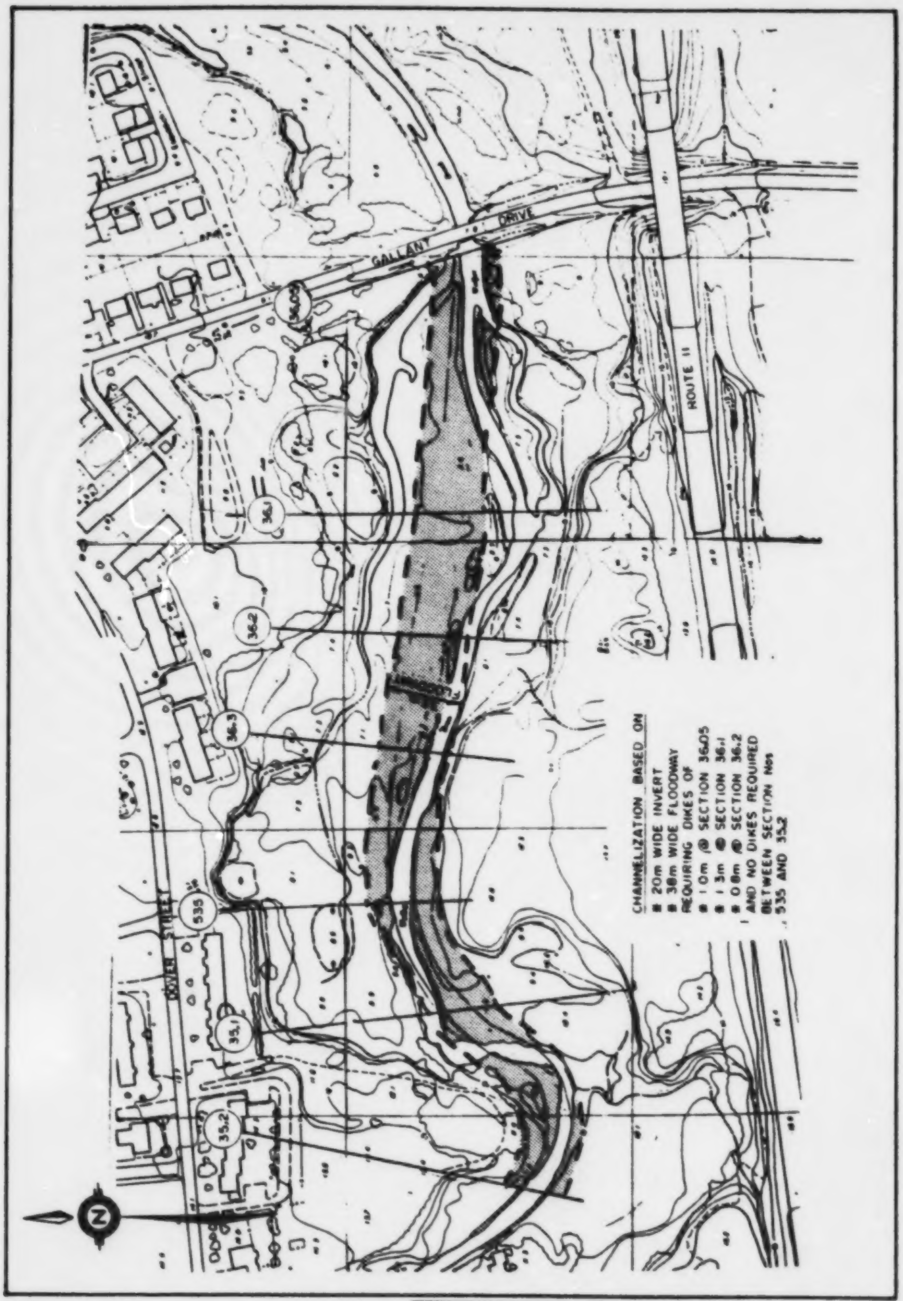
			Return Period Level (m)***	
Location	Section No.		1:20 Year	1:100 Year
Bridge	30.3	Current*	24.53	24.84
		Previous**	24.69	24.88
		(Revised)***	(23.35)	(23.56)
Upstream 50m	30.4	Current	24.32	25.09
		Previous	24.65	25.15
		(Revised)	(24.32)	(25.10)

. No reduction in upstream levels beyond 100m upstream of bridge.

* existing conditions with current brook crossing and flood flows

** flood level with previous flood flow estimates (SMCC, 1985)

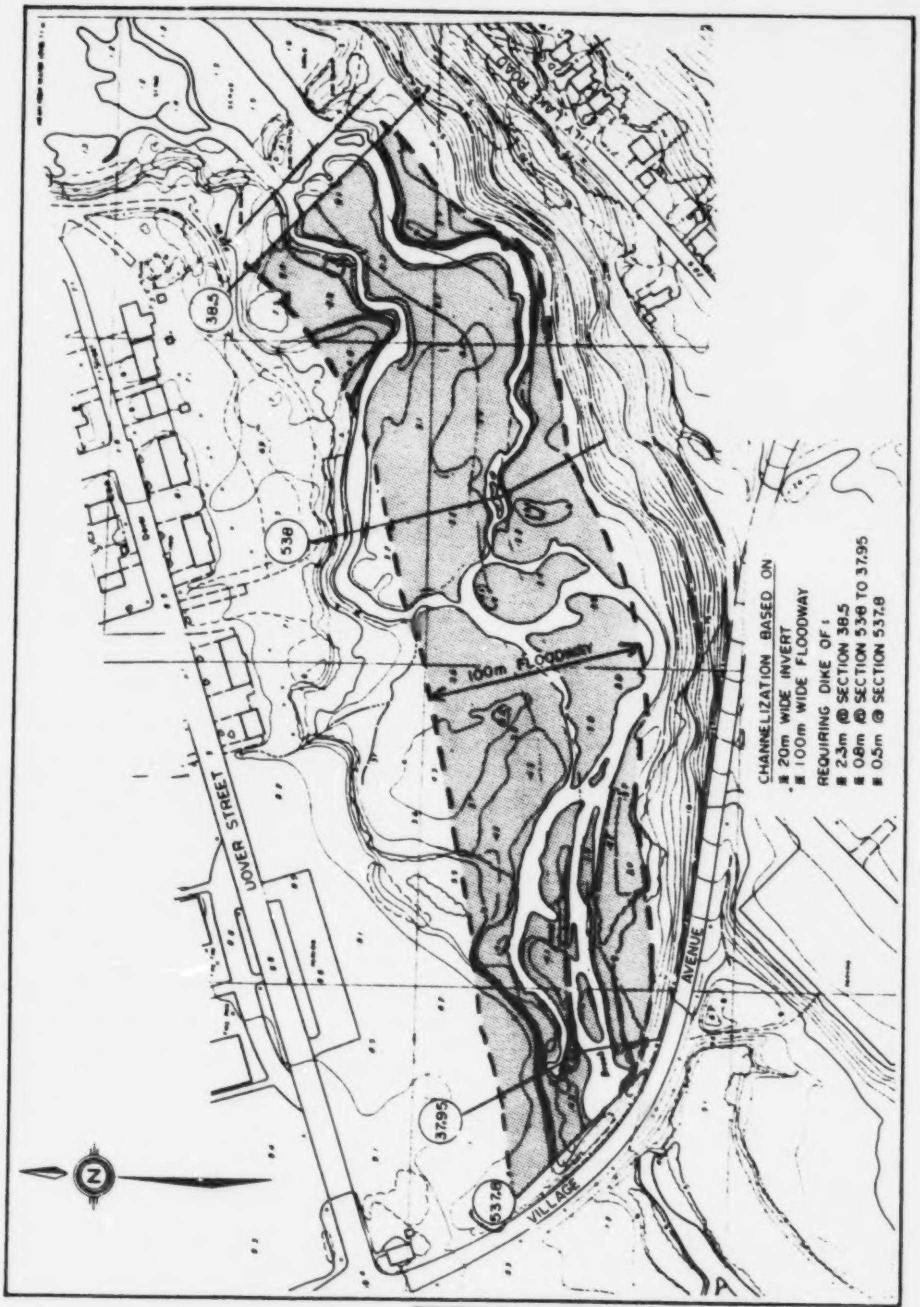
*** flood levels with revised (expanded) cross section or with diked banks



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option:
Channelization - Dover Street
to Gallant Drive
Scale: approx. 1:2100

FENCO FIG. 6-12



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Flood Damage Reduction Option:
Channelization - Downstream
of Village Avenue
Scale: approx. 1:1900

FENCO FIG. 6-13

There are no major engineering works appropriate for this location. Some culvert replacement, and flood proofing (by low berms or elevations on extended foundation walls) are best for this area. These options are costed in Chapter 8 to alleviate problems at the structures discussed in Chapter 4.

6.3 NON-STRUCTURAL ALTERNATIVES

In many flooding situations it is not economically feasible to completely implement structural flood control measures such as those described in the previous Sections. In these cases, however, (and often in association with structural measures) substantial reductions in damages may be achieved through the use of non-structural measures. These may include flood proofing, property acquisition, zoning of flood plain areas to ensure proper development, and the implementation of contingency plans designed to either directly reduce the flood threat (e.g. construction of temporary dikes), or to reduce the resulting damage from a flood (e.g. moving property out of the flood plain).

6.3.1 Early Warning/Contingency Planning Measures

Both long-term plans and emergency plans may be feasible. Long term plans would be those which do not involve a response to a specific potential flood event, but rather involve a state of general preparedness. For Walker Brook it appears that the main threat is from flooding in the spring. Hence, certain residents might be advised to keep valuable property on the upper floors of their residences rather than in basements or on the ground floor, and low basement windows might be sealed during this period of the year. On the municipal government level, a formal program of debris/ice observations at culvert crossings could be instigated together with evaluation of snow surveys and a weather-watch during this critical period.

Short term or emergency plans would be designed to respond to a specific flood threat by the implementation of a planning course of action. Such actions might include evacuation of certain buildings or construction of temporary dikes at critical locations, for example. However, the success of such short term measures depends largely on the receipt of a timely and accurate flood warning linked to a system for rapidly disseminating flood information, so that both communal and individual efforts to reduce flood damage potential could be carried out.

There are, however, a variety of disadvantages to the use of such an early warning/emergency response system as the sole means of controlling flood damages. In particular:

- such systems require sophisticated equipment and operational expertise which would contribute to high, ongoing operations costs;
- the lead time to allow for effective response would be very short because flood flows rise quickly in this relatively small watershed; and
- the successful control of flood damages based upon a flood warning requires rapid co-operation from area residents. Unfortunately, there is evidence elsewhere that warnings are often ignored, and in many cases people may not be available to carry out the appropriate actions.

Hence, it appears that the use of a short term/emergency planning approach will not alone meet the stated objective of controlling flood damage. However, given the general high cost of the structural alternatives, it seems that the development of formal contingency plans may be an effective method of reducing flood damages. Given the difficulties of short-term forecasting for Walker Brook, the emergency plans would have to be designed to handle a flooding situation which had already begun rather than preparing for a predicted event. This could greatly reduce the potential savings in flood damages.

The final details of a flood contingency plan for Walker Brook can only properly be developed by the local officials directly involved in its implementation. However, in the following sections, the main elements which will be required for a plan will be outlined. The main requirements are as follows:

1) a clear identification of those individuals involved in carrying out such a plan, their specific responsibilities and authority vested in them;

2) a detailed description of the lines of communication which must be established between individuals and organizations;

3) a detailed inventory of resources available in dealing with the flood and how these resources can be obtained; and

4) a detailed set of procedures to be followed in the event of an emergency.

Excerpts from a typical flood emergency contingency plan for a municipality are included in Technical Appendix D. The following discussion outlines some of the items of pertinence to Walker Brook.

6.3.1.1 Control of Emergency Operations. In general, an Emergency Operations Control Group should be established consisting of some or all of the following persons:

- the Mayor;
- the City Administrator;
- the City Engineer and/or Director of Public Works;
- the Chief of Police;
- the Fire Chief;
- the Medical Officer of Health;

- the Head of the Social Services Department; and
- a representative of the electrical and telephone utilities.

Upon recognition of a flood emergency, this group (or its designated alternates) should be alerted and should assemble at a pre-defined location (presumably the Municipal Offices). The duties of each member of the group should be clearly laid out in the plan. For example:

- The Mayor would probably have charge of the group and have the power to designate the existence of an official "Municipal Emergency" and to designate an "Emergency Area". He would also have the responsibility of authorizing requests for assistance to Provincial and possibly Federal levels of government.

- Others would be designated in charge of communications, on-site flood fighting and evacuation, traffic control and road closures, establishment of a reception/relief centre, etc.

Within each of these areas of responsibility, the forces available to each member of the operations group would have to be familiar with their role and with the contingency plan. Some form of training in implementation of the plan would be required upon completion of its development.

6.3.1.2 Communications. The establishment of effective communications both within the group responsible for implementing the contingency plan and between this group and the public is a key factor in the success of such a plan. For example, it is conceivable that certain actions designed to minimize flood losses might not be feasible until the electricity had been switched off.

Any delay in contacting the appropriate official would undermine the effectiveness of the plan in reducing losses. The contingency plan should provide details of the following:

- names, addresses and telephone number of: all members of the operation control group and their designated alternates; municipal crews and volunteer groups; Provincial and Federal government officials; and, local radio and television stations through which messages may be relayed

- the establishment of procedures for issuing the flood alert to the affected residents and advising them of emergency procedures which have and will be taken, and a single communication centre to issue reliable reports

- the establishment of communications between the operations control group and the in-field staff, utilizing a short-wave radio system operated by the City. A common link between this system and the Police, Fire and Ambulance services should also be established.

6.3.1.3 Resources. The contingency plan should contain a geographical list of materials, resources and manpower owned, or available to the municipality for use in a flood emergency. This should include vehicles, heavy equipment, sand, barricades, etc. The Technical Appendix shows a typical example.

6.3.1.4 Emergency Procedures. The flood emergency contingency plan should contain details of specific procedures to be followed to minimize flood losses. The areas covered should include:

- flood-fighting procedures, e.g. temporary diking using sand bags;
- evacuation procedures;
- temporary floodproofing procedures; and
- cleanup and re-entry procedures.

For Walker Brook, the following specific points should be considered:

Flood-fighting

The plan should include a map showing the low spots on the river banks at which temporary dykes using sand bags might be erected. In particular, temporary diking would be applicable at the spill area upstream of the Route 11 interchange and along the north bank near the corner of Ortorna and Dover Street, for examples.

Evacuation Procedures

The type of procedures required for evacuation of residents from flooded or soon-to-be flooded areas are well documented in the example flood contingency plan in the Technical Appendix. This can be used as a model for creation of a plan for Walker Brook. The most important elements would be:

- development of a system by which flood alerts can be issued to residents. This should include a priority designation of evacuation and selection of evacuation routes;
- details about the movement of property and vehicles to higher elevations (giving the levels above which property should be safe); and
- the effectiveness of sealing of low windows to prevent entry of water.

Clean-Up Procedures

Upon recession of the floodwaters, certain steps should be followed to ensure safe and sanitary reoccupation of the affected properties. The flood contingency plan should include details about health precautions relating to drinking water, safety precautions (e.g. electrical circuitry), and municipal services (e.g. sewers) which should be checked and reconditioned.

Updating

The final aspect of the flood contingency plan is the need for regular updating of the plan. This should particularly cover personnel, since changes will frequently take place within the organizations likely to be involved. Within this context, review of the plan and training in its implementation by the agencies involved on a regular basis will also be required.

6.3.2 Flood Proofing

Floodproofing techniques are implemented in order to make individual or small groups of flood-prone structures able to withstand the effects of flooding without sustaining damage. Aside from the low berms or floodwalls discussed earlier, two main methods can be used:

- raising of structures (upon fill, piers, extended foundation walls, or piles) to place flood susceptible areas above the design flood elevation. This method generally precludes the use of the basement area for many purposes.

- sealing and strengthening of basements to prevent entry of floodwaters. Sealing of water entry points can also be extended to a point where the flood depth is about 0.5 metres without risking the structural integrity of buildings due to water pressure.

While floodproofing is sometimes applied retroactively for existing buildings, it is generally most economical in new structures.

For the Walker Brook flood plain, where over 100 structures are identified in Flood Risk Mapping as susceptible to flooding at the

100 year return period, a total cost of the order of $800 000 to $1 200 000 would be anticipated for complete implementation of floodproofing. In general, therefore, floodproofing of all flood-prone properties does not seem to be a suitable single solution for preventing flood damages along Walker Brook.

However, the floodproofing of a number of individual properties may improve the feasibility of certain other alternatives. For example, if the option for a berm/floodwall upstream of the Route 11 crossing is selected to eliminate spill into the St. Albert area, then remaining properties which are flood susceptible (i.e. immediately west of the Pine Street crossing and at Dover and Ortorna Streets) could be floodproofed to provide them with the same degree of protection as the remainder of the community. The cost of elevation floodproofing for structures near Pine Street is approximately $63 600. The same for those at Dover and Ortorna Streets is about $78 000.

6.3.3 Property Acquisition or Zoning

Clearly the most natural form of flood protection available is to avoid development within the limits of a river's flood plain. Alternatively, given that development has taken place, the acquisition and removal of flood-prone properties will serve the same purpose. Unfortunately, such an alternative is not always feasible because of the high cost, the social disruption, the potential loss of tax base, and the general pressure for developable land. For the Walker Brook case, it would cost several million dollars to acquire all properties located within the 100 year flood plain, and to restore the area to natural form. In addition to this high cost, it appears certain that such a solution would be politically impossible to implement.

This disadvantage does not mean, however, that selective acquisition of certain of the most flood-prone properties should not be considered to assist the overall implementation of other alternatives, - when/if they come on the market at affordable rates. Otherwise, floodproofing will be the most economical route.

Zoning of flood plain areas to restrict development is a frequently applied procedure. This policy is well founded as it will prevent the increase of flood damage potential, and it is suggested that the restricted area be adjusted to agree with the current, estimated 1 in 100 year flood plain. However, such a policy will not reduce the damage potential for existing development, and hence cannot be considered as a comprehensive flood damage reduction measure.

7.0 ENGINEERING & ENVIRONMENTAL CONSIDERATIONS

7.1 GENERAL

As noted in the Terms of Reference (TOR), the primary purpose of this study was to focus on structural measures which directly affect the flood characteristics. However, to do so, it was imperative that the preliminary structural measures being considered as flood damage reduction options for subsequent refinement, cost estimates, and benefit-cost analyses, first be assessed in terms of some basic planning considerations, both engineering and environmental.

The engineering considerations are viewed in a planning context of three subject areas; municipal, transportation, and cost. This chapter will address the municipal and transportation engineering considerations, while Chapter 8, which follows, will present the basis of the engineering cost estimates and a sample of a total estimated cost calculation for one structural alternative.

The environmental considerations are viewed in terms of social implications and impacts, as well as the physical environment and regulations relating thereto. In particular, the environmental considerations relate primarily to positive or negative impacts arising from the implementation of a particular structural measure.

As discussed in the previous Chapter 6, the remedial requirements for the upstream Val D'Amour area are relatively minor and straightforward, and do not involve a choice and evaluation of basic approaches as is the case for that portion of the flood plain downstream of the Sugarloaf Provincial Park. Since the engineering considerations involve more variables and the environmental sensitivities (social and physical) are both greater for the flood plain downstream of Sugarloaf Provincial Park, this chapter will concentrate on this portion of the study area.

7.2 BASIC OPTIONS DOWNSTREAM OF SUGARLOAF PROVINCIAL PARK

In essence, there are two basic options available for remedial structural measures applicable to that existing five kilometre portion of the Walker Brook flood plain downstream of Sugarloaf Provincial Park:

a) one involving minimal encroachment on existing development, by a substantial diversion of flood flows, including:

   i) construction of berms and improvements near Sugarloaf Provincial Park; and
   ii) construction of a controlled flow diversion of floodwater at the existing Walker Brook/Route 11 point of crossing and a floodflow diversion channel along the south side of existing Route 11 to a new Route 11 crossing some 1.25 km downstream, and re-entry to existing Walker Brook (with or without channel improvements downstream of this point of re-entry)

   or,

b) one involving considerable encroachment on existing development, by structural improvements along existing Walker Brook, including:

   i) construction of berms and improvements near Sugarloaf Provincial Park (i.e. as per item a) i) previous);
   ii) provision for increased capacity at the existing Walker Brook point of crossing of Route 11, involving installation of an additional culvert, and channel grading, provision of erosion protection, and construction of a combined headwall at the inlet end of the improved crossing; plus
   iii) numerous and various types of structural options downstream of this enlarged waterway crossing (with or without channel improvements downstream of the Dover Street Lower Crossing).

Basic Option a) would divert 40 m^3/s of the estimated 1:100 year return period flood flow of 62.7 m^3/s around some 2.1 km of the existing/partially developed flood plain; whereas basic Option b) would implement structural alternatives along this same 2.1 km of the existing flood plain sufficient to accommodate the complete 62.7 m^3/s flood flow.

7.3 ENGINEERING CONSIDERATIONS

In addition to the hydrologic and hydraulic analyses presented in Chapters 3 and 4 previous, the initial development, selection, and refinement of structural alternatives requires the application of engineering and planning principles involving:

a) municipal services: including impacts to existing infrastructure and, where identified, sub-area storm drainage requirements that currently exist and will continue to exist, independent from remedial measures considered or recommended as a result of this Walker Brook flood plain study; and future implications to such things as curb and gutter and sidewalk development, where these facilities don't already exist at the time of implementation of some of the specific types of structural alternatives (i.e. in the normal staged development of improved municipal services); and

b) transportation facilities: on the macro scale, involving planning by the Provincial Department of Transportation of the ultimate development of the Campbellton By-pass portion of Route 11 to a future 4-lane divided highway; and, on a municipal level, the City's ultimate development of its primary city streets (eg., Val D'Amour Road Diversion to arterial street status, and Dover Street and Village Avenue to collector street status, etc.).

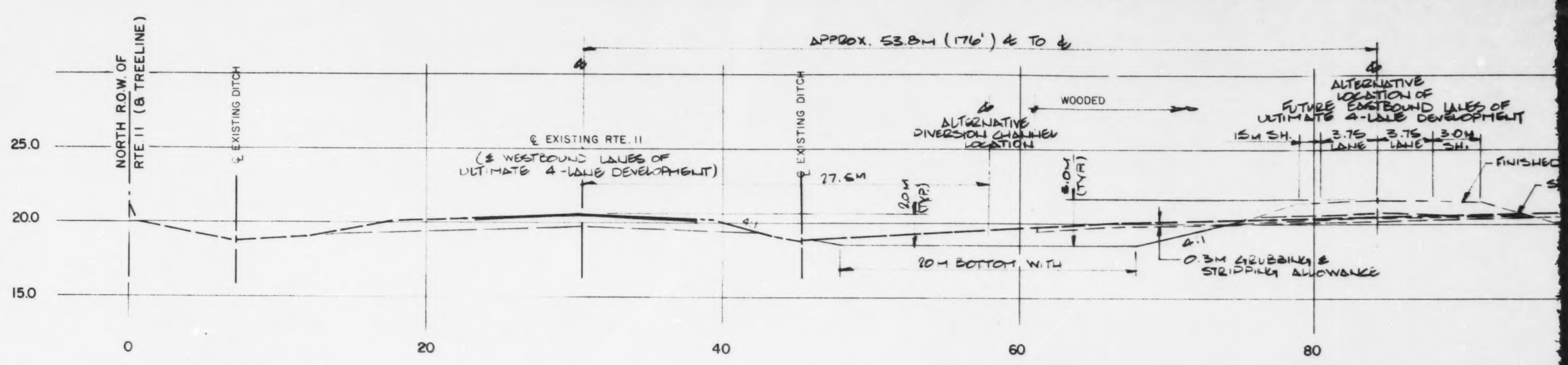
Since Basic Option a) of sub-section 7.2 preceding (i.e. that of a diversion channel along the south side of existing Route 11 to take flood flows) has the advantage of minimal encroachment to existing development along the flood plain, and would require the approval and joint planning of the Department of Transportation, the subject of transportation engineering considerations will be addressed first.

7.3.1 Transportation

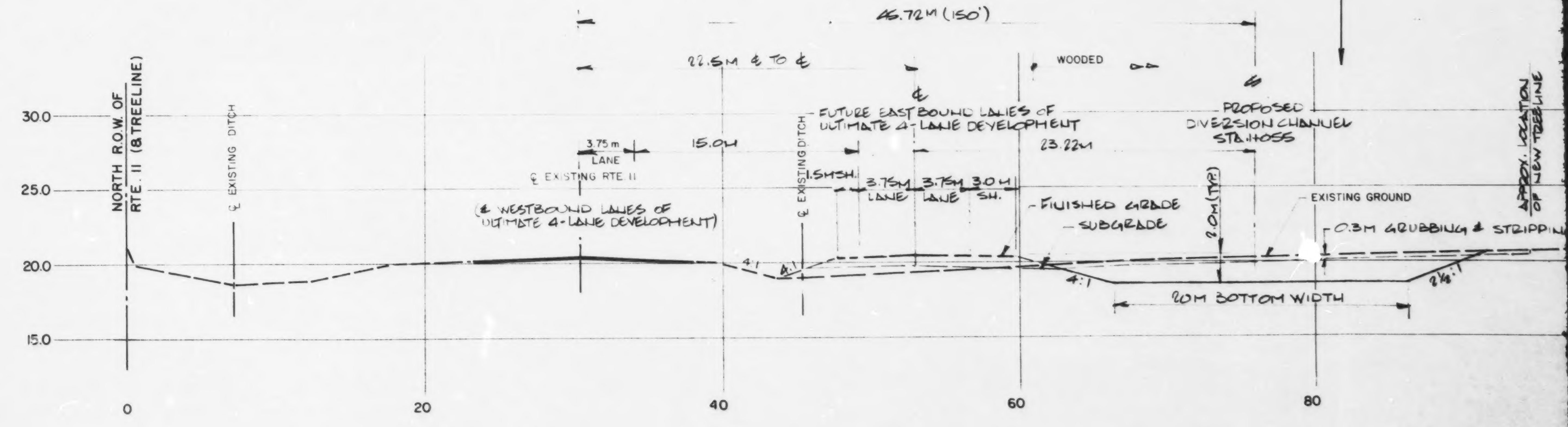
7.3.1.1 Provincial Implications. When the Department of Transportation planned the Campbellton By-pass portion of Route 11 in the 1960's, it was planned as a controlled access highway and for its ultimate development to a 4-lane divided highway; and in fact sufficient right-of-way was acquired by the Department at that time to enable the future/second stage development on the south side of the initially developed 2-lane facility. Although there is no immediate necessity nor plans to proceed with the second stage of the 4-lane development, the Department has successfully maintained this area of Route 11 free of accesses, and the additional southside right-of-way is still available and without restrictions for development of the second stage as and when the Department requires.

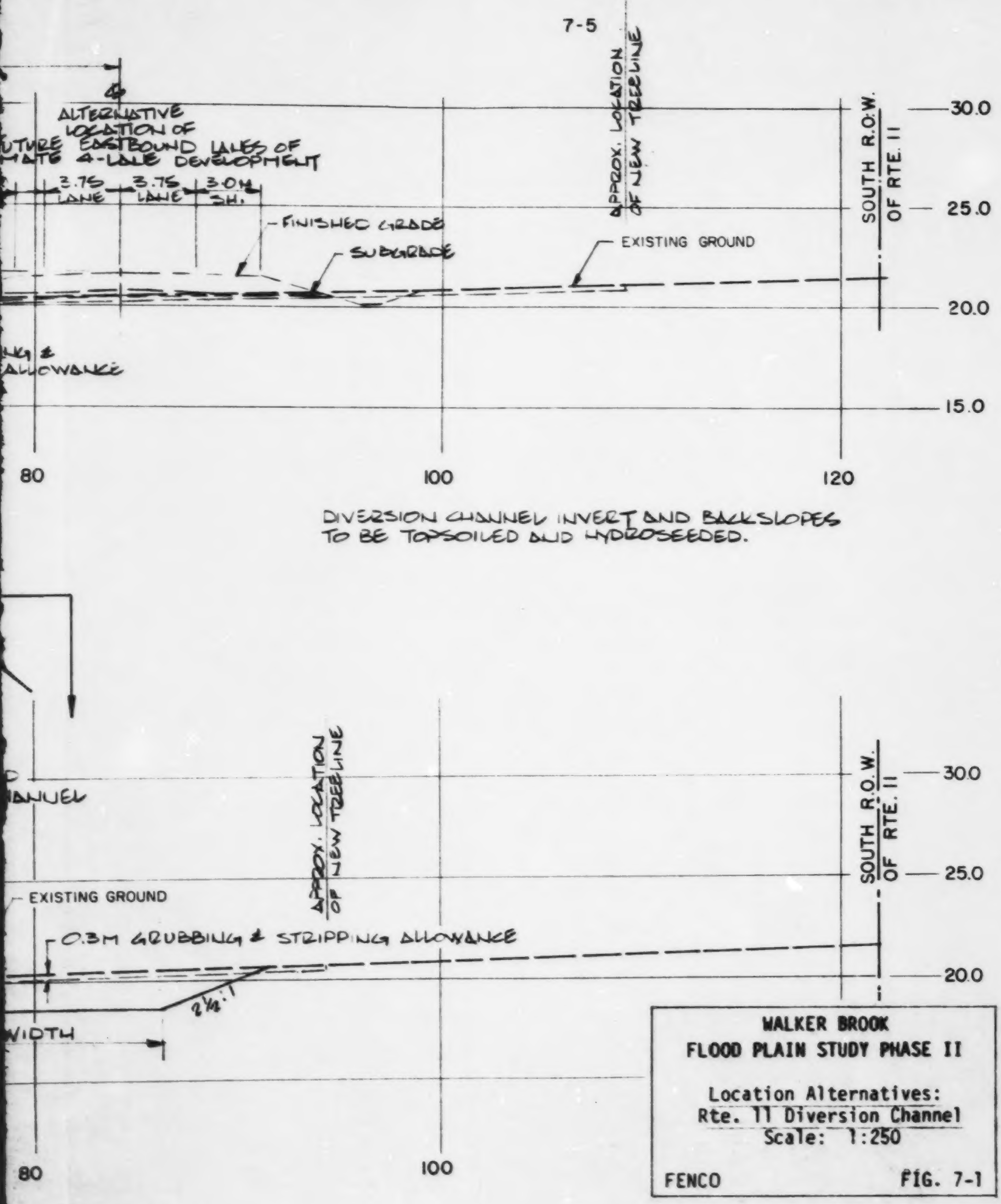
Due to the importance of this provision in the context of Route 11 and the Department's long range transportation planning, it is imperative that any consideration of joint-use of this southside right-of-way be compatible with the Department's ultimate 4-lane development plans, and completely acceptable to it. As a result, considerable attention was given to this matter in this study, particularly with regard to the most appropriate location of a 20 m bottom-width open channel, as required to accommodate the projected 1:100 year return period flood flows, relative to the Department's ultimate development cross section for a 4-lane divided highway.

Figure 7-1, which follows, illustrates two location alternatives of a 20 m wide diversion channel relative to future development of this area of Route 11 to a 4-lane divided highway. Both


APPROX. 53.8M (176') ℄ TO ℄
℄
NORTH R.O.W. OF RTE. 11 (& TREELINE)
℄ EXISTING DITCH
℄ EXISTING RTE. 11
(℄ WESTBOUND LANES OF ULTIMATE 4-LANE DEVELOPMENT)
℄ EXISTING DITCH
27.6M
2.0M (TYP.)
℄ ALTERNATIVE DIVERSION CHANNEL LOCATION
WOODED
8.0M (TYP.)
℄ ALTERNATIVE LOCATION OF FUTURE EASTBOUND LANES OF ULTIMATE 4-LANE DEVELOPMENT
1.5M SH.
3.75 LANE
3.75 LANE
3.0M SH.
FINISHED
4:1
4:1
20M BOTTOM WITH
0.3M GRUBBING & STRIPPING ALLOWANCE
25.0
20.0
15.0
0
20
40
60
80

RECOMMENDED ALTERNATIVE:
DIVERSION CHANNEL RELATIONSHIP
TO FUTURE 4-LANE DEVELOPMENT





WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Location Alternatives:
Rte. 11 Diversion Channel
Scale: 1:250

FENCO
FIG. 7-1

alternatives can be developed within the existing southside right-of-way, and the hydraulic requirements of the open channel are sufficient for the 1:100 year return period design flood flow of 40 m^3/s in each instance.

The upper cross section alternative of Figure 7-1 locates the diversion channel between the existing 2-lane highway (the westbound lanes of the ultimate 4-lane development) and the future/second stage development. An advantage of this alternative is that an initial development of the diversion channel would be immediately adjacent the existing highway, thus minimizing the visual effect of the diversion channel development within the existing right-of-way (i.e. relative to the lower cross section alternative). Disadvantages of this alternative are:

- to the developer of the diversion channel: a greater construction cost for the new, piped, structural crossing toward the east end of the diverison channel, due to the alignment constraint at the entrance to the piped crossing; and
- to the Department of Transportation: greater grading quantities, and thus higher construction costs, for their ultimate development of the two eastbound lanes, due to the steeper sidehill condition at the south side of the right-of-way provision.

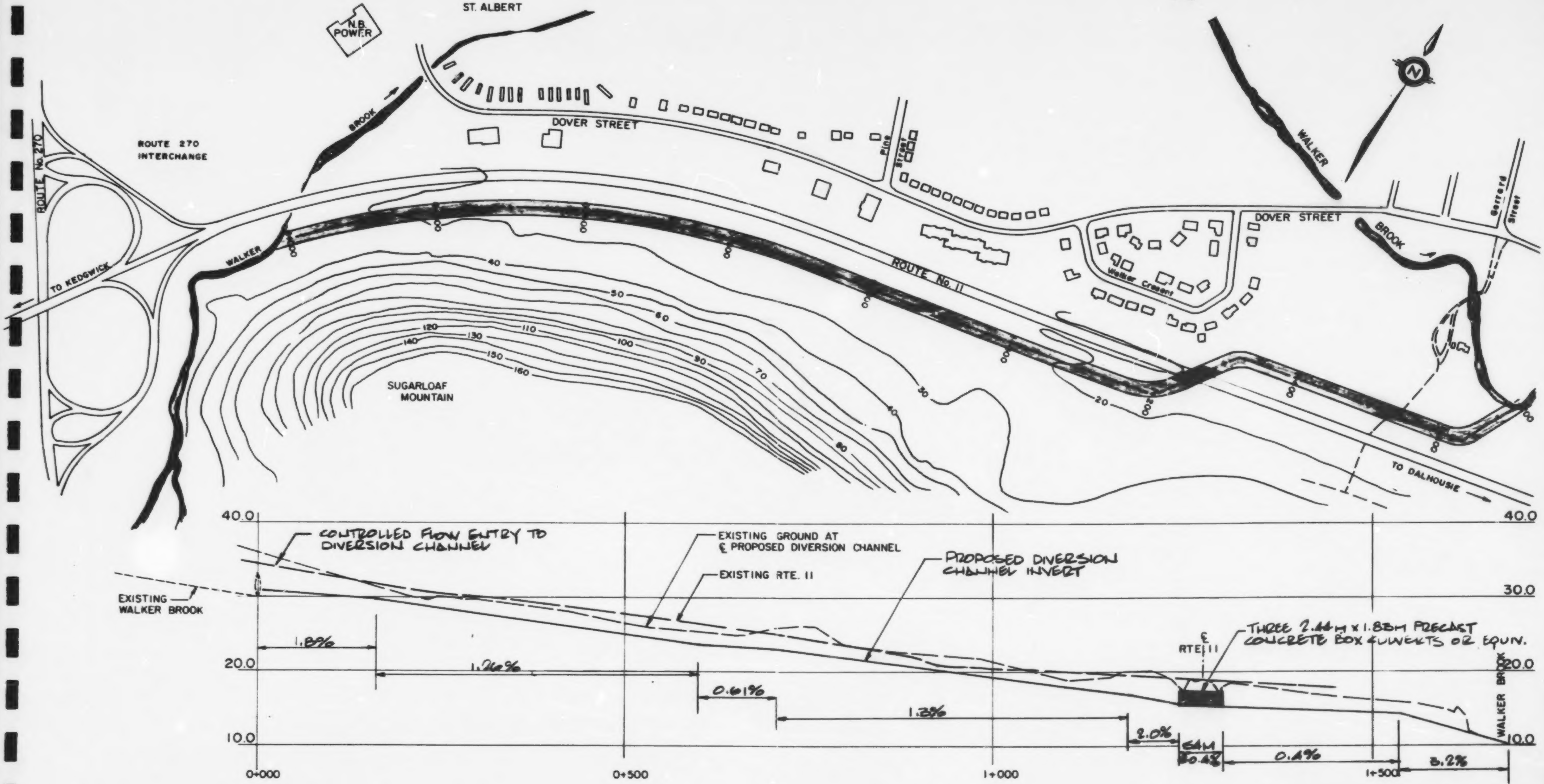
As will be noted from Figure 7-1, even with an independent profile for a proposed finished gradeline of the future eastbound lanes, the combined lateral requirement of the two future developments (i.e. diversion channel plus second stage highway) is approximately 7 metres greater for the upper location alternative illustrated.

The lower cross section alternative of Figure 7-1 locates the centreline of the second stage of the 4-lane highway development at 22.5 m from the centreline of the existing highway, in conformance with the Department of Transportation typical 4-lane development cross section using a "narrow open median" (i.e. 15.0 m between the painted edge lines of the two 2-lane roadways), and locates the diversion channel on the south side of the right-of-way. The only disadvantage of this alternative is a visual one, whereby an approximate 33 m width of the existing wooded right-of-way would have to be cleared in order to enable the initial construction of the diversion channel, without any apparent reason to a layman as to why this was necessary (i.e. with the undeveloped area of the future eastbound lanes remaining between the existing highway and the diversion channel).

It is concluded that this latter disadvantage is more than offset by the cost advantages of this alternative to the Department of Transportation, without whose approval such a diversion channel development would not be possible in any event. Thus Figure 7-1 notes this alternative as the recommended cross section for ultimate development, and it is this diversion channel location that is illustrated in Figure 7-2, Plan & Profile: Recommended Rte. 11 Diversion Channel, which follows.

It is obvious that this recommended cross section alternative, and in fact the whole subject of a joint-use right-of-way along this portion of Route 11, will have to be subject to detailed discussion with the Department of Transportation at a very early stage, and certainly before finalization of Phase II of this subject.

Figure 7-2 illustrates that approximately 1.25 km of the overall 1.7 km length of the Rte. 11 diversion channel is located on the south side of the existing highway. The necessity to cross the existing highway at this point was due to steeper sideslopes east of Sta. 1+200; which, utilizing either of the Figure 7-1 development cross section alternatives, would have resulted in prohibitive grading quantities (and thus construction costs), and a requirement for acquisition of additional south side right-of-way. Since there is no existing development along the north right-of-way



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Plan & Profile: Recommended
Rte. 11 Diversion Channel
Scale: approx. 1:5 000 H
1:500 V

FENCO

FIG. 7-2

limit in this area, and neither would there be any access permitted from Route 11 to any proposed development, the least cost alternative is to cross from the south to the north side of Route 11 at approximately Sta. 1+250 and acquire the necessary right-of-way, under this project, to develop the remaining 400 m of diversion channel outside the existing north limit of the Route 11 right-of-way.

7.3.1.2 City Implications. City implications evolving from transportation engineering considerations arise entirely from Basic Option b) (i.e. structural improvements along existing Walker Brook), from the City Limit just west of the existing Walker Brook point of crossing of Route 11 to, and including, Village Avenue at the east. In all instances where a structural measure involved a City street designated for ultimate development to an arterial or collector street standard, as designated on the City's Municipal Land Use Plan, Schedule 'C', this was taken into account when developing the location details of particular structural measures; eg., where designated streets have not yet been developed to a curb and gutter cross section with parking lanes and a sidewalk, or sidewalks, the spacial requirements for these developments were allowed for when establishing the locations of berms, and hence the impact of the latter on existing driveways, frontage fences, private lot improvements, etc.

In one instance, that of the Department of Transportation's preliminary plans for development of a new Walker Brook Bridge No. 3 at Village Avenue, a City street designated as a collector, both City and Provincial considerations are involved. In this instance, the Department's "Preliminary" construction details were used for engineering purposes of hydraulic analysis as it pertained to this study; however, no estimated construction cost was carried for this proposed replacement structure in the estimating of Flood Damage Reduction Costs, of Chapter 8, as this was considered as a prior Provincial commitment.

7.3.2 Municipal

7.3.2.1 Drainage to St. Albert Area Independent of Walker Brook. In the detailed evaluation of the origin and magnitude of "spill flows" affecting the floodprone developed areas of the St. Albert Area, it was determined that there are three topographic catchment areas that, quite independent of Walker Brook-generated spill flows, can contribute significantly to flooding in the St. Albert Area. These three catchment areas are illustrated on Figure 7-3, which follows, and include approximately:

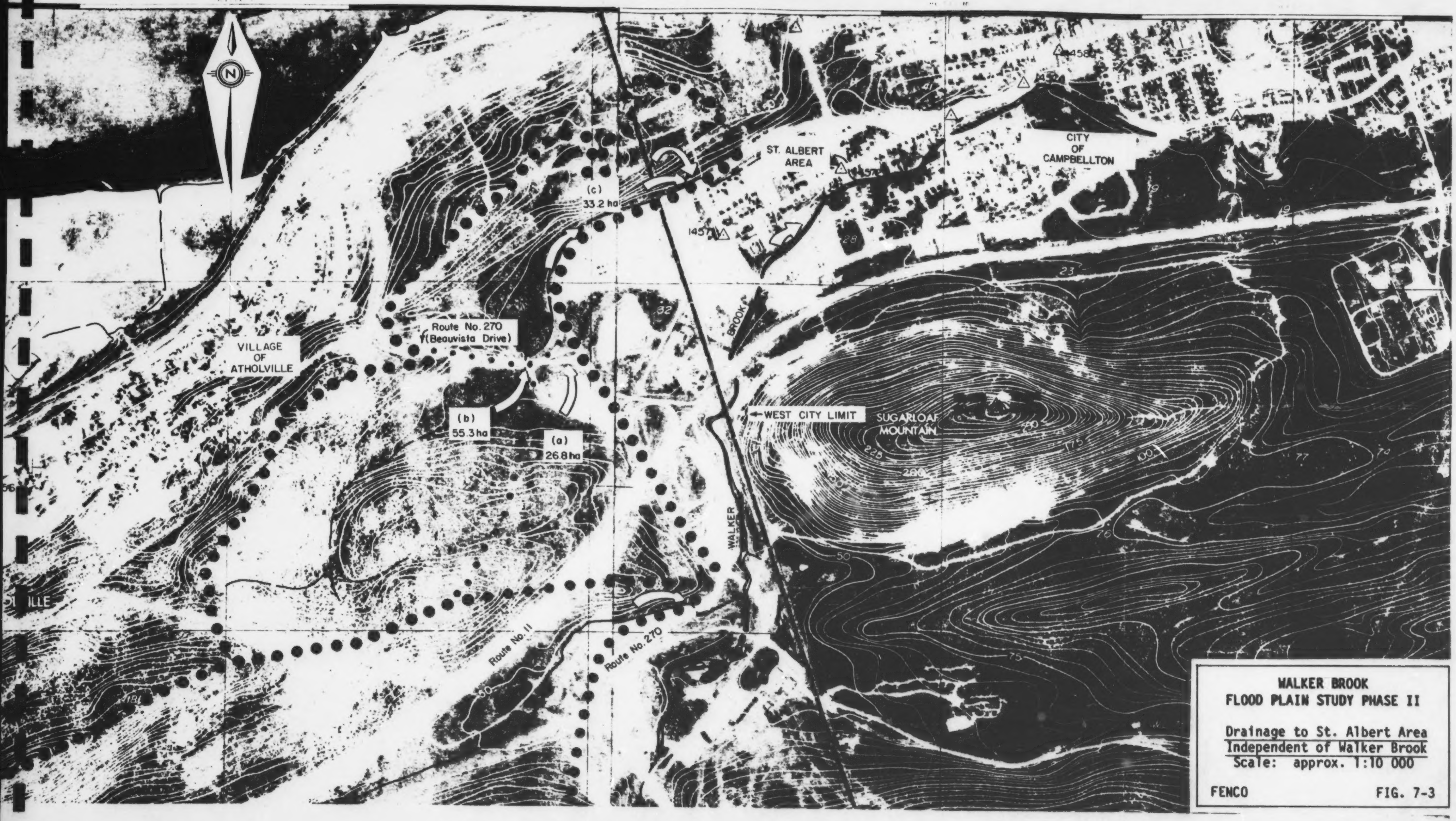
a) 26.8 ha in the catchment area immediately to the west of the Route 11/Route 270 interchange, and draining to the at-grade intersection of Route 270 west (Beauvista Drive) and Val D'Amour Road Diversion;

b) 55.3 ha in the steep mountainous catchment area immediately to the southwest of the at-grade intersection of Route 270 and Beauvista Drive (and also draining to this point, as per a) above); and

c) 33.2 ha in the steeply sloping sidehill area immediately to the north and west of Val D'Amour Road Diversion, between its at-grade intersections with Beauvista Drive and Dover Street.

Depending on frozen ground conditions, the times of concentration of these overland flows can be expected to be very minimal off these steeply sloping areas, resulting in high runoff rates to the flat sideditch along the west and north sides of Route 270 below. Depending on the operational capacity of this ditch, culverts, and storm drains, and their reduced capacities when partially filled with ice and snow, it is concluded that these three adjacent catchment areas contribute significantly to "spill flows" into the adjacent developed areas of St. Albert.

A detailed condition survey of existing storm drainage facilities



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Drainage to St. Albert Area
Independent of Walker Brook
Scale: approx. 1:10 000

FENCO FIG. 7-3

downstream of these three catchment areas is beyond the scope of this study, as this is essentially a municipal storm drainage matter. However, it is recommended that this finding be brought to the attention of the appropriate authorities of the municipalities of Atholville and Campbellton; since, without a storm drainage system designed to accommodate the overland flows from these three catchment areas, the implementation of recommended remedial measures arising from this study will not completely resolve the problem of "spill flows" into the developed St. Albert area.

In conjunction with preliminary work for the subsequent Chapter 8, Flood Damage Reduction Costs, an order-of-magnitude estimate was made of the cost to provide: an additional culvert crossing on Val D'Amour Road Diversion, in order to split flood flows to the north and south side open ditches; improvements to approximately 425 m of each of these open ditches, together with the construction of a nominal ½ m height berm outside approximately 200 m of the south side ditch; plus a combined culvert and storm sewer on Alexander Street which would discharge to Walker Brook. The total estimated cost of these works, to accommodate overland flows from these three independent catchment areas, would be in the order of $250 000. (This figure would be subject to refinement upon completion of detailed field surveys and preliminary engineering design based on same.)

7.3.2.2 Typical Municipal Services Considerations. In the development of remedial structural measures, and costing of same, considerations were given to the following, - particularly with regard to the development of dikes along selected reaches of Walker Brook and berm-type protection of floodprone properties (both of which are an essential requirement of Basic Option b), involving structural improvements along existing Walker Brook, only):

a) increased depth of cover over: existing systems such as water distribution, and sewage and stormwater collection systems; private lot services; and the necessity to raise components of these subsurface utilities in order to match higher elevations in conjunction with implementation of specific structural measures;

b) reduced clearances to aerial utility lines, and the necessity to relocate or raise lines that are adversely affected by specific structural measures, in order to maintain minimum Code-specified clearances from new/higher ground levels;

c) revisions to surface drainage patterns on individual private lots, or particular groups of lots; and

d) maximum practical gradients of private driveways, so as not to create winter access difficulties.

From a municipal services viewpoint, structural measures involving the development of dikes and protective berms, whether on existing street right-of-ways or on adjacent private property, create some very real and practical difficulties in implementation, and these have been taken into account in the estimation of flood damage reduction costs of Chapter 8. In addition, the social impacts of these particular measures will be discussed further in Section 7.4.1.

7.4 ENVIRONMENTAL CONSIDERATIONS

This section will briefly discuss the implications of implementation of a program of remedial structural measures as it would affect the social environment of the area, as well as the physical environment.

7.4.1 Social Impacts

Basic Option a), involving the controlled flow diversion of floodwaters to a new open channel along Route 11, will have a minimal adverse impact to existing properties and property owners along the developed and floodprone portion of the Walker Brook flood plain, since the majority of the construction would be far-removed from them.

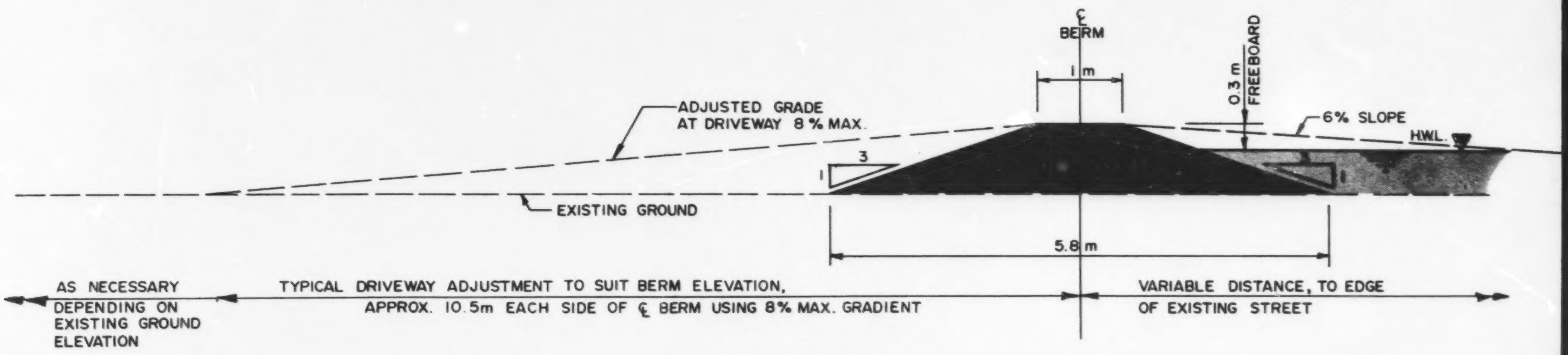
Conversely, Basic Option b), including development of remedial works such as dikes and berms completely within the existing flood plain, would have a severe impact on existing properties and property owners. See Figures 7-4 and 7-5, which follow, that illustrate typical berm cross sections of nominal ½ m height (for built-up areas) and 1 m height (for undeveloped areas and dikes).

First, it is assumed that the details of the proposed program would have to be subject to an approval in principle process, conducted jointly with the developing authority, the affected municipality, and the general area property owners that would be affected, and it is assumed that the mechanism for this process would be some form of municipal public hearing.

Then, once this approval in principle is established, including whether or not power of expropriation would be available to the developing authority for purposes of program implementation, in the common good, it would be necessary to establish working easements and lot-specific agreements with each individual property owner where remedial work has to be performed on private property. These would include agreement on such lot-specific details as removal, relocation, or replacement of private fences, shrubs and trees, planting beds, private walkways, clothesline poles, and the like, as well as the specifics of revised surface drainage patterns and reconstructed driveways, as noted in Section 7.3.2.2 previously.

In such instances, the complete co-operation and advance agreement of several adjacent property owners may be necessary in order to proceed with implementation of a particular structural measure for an area, - a very real weakness of Basic Option b).

Due to this program vulnerability aspect of Basic Option b), the



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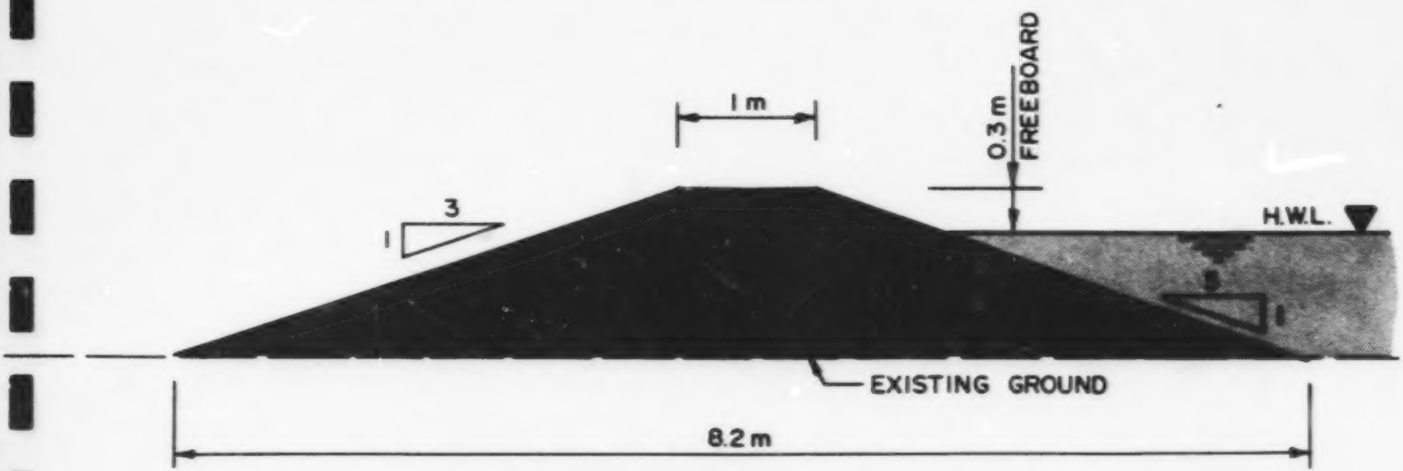
FLOOD PLAIN STUDY PHASE II

Typical Berm X-Section
Nominal ½ m Height
(for built-up areas)

FENCO FIG. 7-4



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Typical Berm X-Section
Nominal 1 m Height

FENCO

FIG. 7-5

alternative Option a) offers many practical advantages of implementation.

7.4.2 Environmental Impacts

Practically all aspects of any remedial program, whether involving Basic Option a) or b), will be subject to the regulatory requirements of the Provincial Watercourse Alteration Regulation 77-7, a regulation under the Provincial Clean Environment Act, as well as the Federal Environmental Assessment Review Process (EARP) and Fisheries Act. These regulations will govern the timing, methods of construction, and thus, indirectly, the cost of all on-site work.

However, the degree of difficulty to be addressed with Basic Option a), the Route 11 Diversion Channel, will be considerably less than that associated with Basic Option b), where all construction is located in the existing Walker Brook flood plain.

The extent of excavation and subsequent topsoiling and hydroseeding of excavated slopes is considerably greater in the case of Basic Option a); however, the work area can be isolated from affecting the water quality of Walker Brook until such time as erosion-free surfaces are established, and thus an important environmental concern of this construction program may be effectively mitigated.

Two structural measures identified in the earlier Phase I report of this project, and in the Terms of Reference for this study, included:

- removal of flow obstructions, including an abandoned dam downstream of Village Avenue; and

- additional channel improvements such as clearing vegetation, dredging, straightening, and enlargement.

In particular with respect to the latter channel improvements:

- Section 6.2.5.5 previously noted that there were no damage reduction benefits to be derived from such channel improvements between Dover Street and Gallant Drive, but also noted that the area of floodprone land could be reduced by development of a channel and dikes along both sides of the channel, as illustrated on Figure 6-12; and

- Section 6.2.5.6 previously noted similarly with regard to channelization downstream of Village Avenue (this area including the ruins of the old dam), but also noted that the size of the flood risk area could be reduced by a channelization design such as shown in Figures 6-9 and 6-13.

  The estimated costs were developed in Chapter 8 for such channelizations, Dover Street to Gallant Drive and downstream of Village Avenue; however, in view of the fact that the projected floodwater forecasts for these two areas do not indicate any potential damage to existing structures along these two reaches, and due to the considerable estimated construction cost for the channelization of those two reaches (at $150 000 and $315 600 respectively) with no tangible economic benefits resulting, it was decided to assess the other considerations that would be involved with such extensive grading, diking, and slope restoration work.

Thus for one advantage, improved hydraulics resulting in a reduced flood risk area, the associated disadvantages of channelization downstream of the Lower Dover Street/Gerrard Street area are as follows:

a) a large cost (at a combined estimated cost of $465 600) without any economic benefit;

b) since this channelization would be either in or immediately adjacent the existing meandering channel, the utmost of care would have to be exercised during the construction phase to ensure that the waterway is not subjected to siltation from erosion of excavated or embankment slopes, meaning that large settling ponds would be required until such time as fully grassed surfaces are established on all constructed surfaces;

c) the channelization work areas would be highly visible from the higher grounds along each side of the valley and, as a result, notwithstanding the very real concerns that would exist and have to be addressed to minimize negative impacts to aquatic life, flora, and fauna, there would be the speculative concerns that would be difficult, if not impossible, to satisfactorily allay. (It's understood that there already exists an ecological sensitivity about the lower portion of the flood plain, in the area considered by the Department of Transportation in one of their functional planning alternatives to relieve an existing problem in the Lily Lake Interchange to Route 134/Ramsay Street area); and

d) at some future time, there could prove to be a residual value in what has been referred to as the "old dam ruins" at the approximate mid-point of the Lower Dover Street to Ramsey Street segment of the reach; and what may appear to be an unsightly hydraulic constriction to a hydraulic engineer, to be removed to improve the hydraulics of floodwater flows, could very well be seen as the basis of a theme for a park development to another party.

In summary, based on a fuller assessment of all the considerations, it is the recommendation of this study that channelization of these two segments of the flood plain not be considered at this Phase II stage of the project.

8.0 FLOOD DAMAGE REDUCTION COSTS

The flood damage reduction options described in Chapter 6 were examined in detail to determine the most cost-effective solutions, given the circumstances of:

a) proximity of existing private property and development, and the resultant warrants for aesthetically and socially acceptable solutions;

b) the environmentally sensitive nature of the existing watercourse and its tributaries;

c) the regulatory requirements of the Federal and Provincial environmental and fishery acts and regulations, which will govern the timing and methods of construction of all on-site work;

d) the highly visible nature of any remedial works along the existing flood plain from higher grounds along both sides of the valley;

e) the presence of both aerial and underground public utilities (eg., poles with aerial lines at minimal clearance above existing ground, and private water curbstops, both of which will require modifications due to the construction of dikes and berms);

f) the construction standards and the long term planning requirements of the Department of Transportation in areas where the proposed measures affect either existing or planned Department facilities; and of course

g) the necessity for application of sound engineering principles in the planning and conceptual design of these long term civil works.

The whole basis of the benefit-cost analyses portion of this study is of course very dependent on the estimation of realistic capital costs for construction of the various structural alternatives under consideration. In the development of these cost estimates, the following approach has been taken:

- order-of-magnitude quantites have been estimated for the various items of work or, where appropriate and a sound estimating base has already been established, per lineal metre or per square metre units have been used;
- current construction costs have been used, based on a considerable background of construction cost estimating for N.B. Department of Transportation projects; and
- all alternative-specific cost estimates are all-inclusive, in that appropriate allowances have been included for the applicable non-construction costs as well; such as separate material supply, utility removal and relocation, nominal allowances for working easement privileges on private properties (for the construction of berms and dikes), etc.

In the case of the larger, bridge structure items, estimates have been based on the average results of the three lowest bidders on comparable publicly tendered Contracts. In all instances, to a sub-total of the total estimated construction cost for an alternative, an allowance of 20% has been added for related engineering and contingencies costs.

All cost estimates are based on May 1988 dollars.

Table 8-1, which follows, summarizes the total estimated capital cost to develop each of the structural alternatives; and Section 8.1 presents a sample total estimated cost calculation of a particular structural alternative.

TABLE 8-1

ESTIMATED CAPITAL COST OF STRUCTURAL ALTERNATIVES (in May 1988$)

NOTE: the option titles and numbering sequence used corresponds with the options discussed in Chapter 6, Flood Damage Reduction Options.

ITEM NO./OPTION	ESTIMATED COST
6.2.1 St. Albert Spill Area	
i) Spill from the main channel	$ 14 400
ii) Spill from the western tributary, sub-watershed area #130	24 000
iii) Spill from the area west of the drive-in theatre	42 000
(Thus, total this option only = $80 400, for the option referred to hereafter as the Sugarloaf Park Area Structures)	
6.2.2 Spill Area Upstream of Culvert Crossing at Rte. 11 (i.e. Diversion along Rte. 11)	$623 300
6.2.3 Bridge Expansions	
6.2.3.1 Dover Street (Lower Crossing)	$738 000
6.2.3.2 Pine Street	374 400
6.2.3.3 Dover Street (Upper Crossing)	670 800
6.2.4 Flow Obstructions	
i) Remove abandoned dam below Village Avenue	$ 50 400
ii) Increase capacity at existing Rte. 11 Crossing	$120 000
6.2.5 Diking and Channelization	
6.2.5.1 Diking - between Rte. 11 and Dover Street (Upper Crossing)	N/A
6.2.5.2 Diking - above Pine Street	$ 63 600
6.2.5.3 Diking - at Dover Street (Lower Crossing)	78 000
6.2.5.4 Channelization - Alexander Street*	40 800
6.2.5.5 Channelization - Dover Street to Gallant Drive	150 000
6.2.5.6 Channelization - Downstream of Village Avenue	315 600
6.2.6 Val D'Amour Area	$ 6 000

* includes structural modifications to the footbridge.

8.1 SAMPLE TOTAL ESTIMATED COST CALCULATION OF A STRUCTURAL ALTERNATIVE

The following is a sample calculation of the cost of a specific structural alternative, the Section 6.2.2 Item "Spill Area Upstream of Culvert Crossing at Route 11 (i.e. Diversion Along Route 11)", hereinafter referred to as the Route 11 Diversion Channel.

a) Works at inlet end of existing 5 050 mm (S) x 3 330 mm (R) SP CSP Pipe-Arch to control entry of flood flows to Route 11 Diversion Channel, including minor clearing and grubbing and common excavation, foundation excavation (assume earth), approx. 15 lin. m of gabion weir, construction of a concrete headwall at inlet end (to develop an entrance headloss coefficient Ke of 0.5 max. to effectively utilize its potential existing capacity), and supply and placing of backfill:
estimate combined lump sum value of work = $10 000.

b) Construct new open channel diversion to take $Q_{100}-Q_2$ = approx. 40 m^3/s overflow so as to bypass the developed area. Total length of diversion, including Rte. 11 x-ing (@ 54 m) = approx. 1 700 m.
From quantity take-off sheets:
. Clearing and grubbing = 6 ha
:. 6 ha @ combined est. cost of
$1 400 + $3 000 = $4 400/ha = $ 26 400
. Common excavation = 65 000 m^3
:. 65 000 m^3 @ $2.50/$m^3$ = $162 500
. Topsoil = 4 500 t @ $5.00/t = $ 22 500
. Hydroseeding = 2 ha @ $5 000/ha = $ 10 000
:. Est. cost of open channel only = $221 400
say $222 000

c) New culvert x-ing of Rte. 11:

Use three culverts, each 2.44 m (S) x 1.83 m (R) x 54 m (L), precast concrete units, where mat. supply costs	= $186 000
Foundation excavation (earth) = 2 400 m^3 @ $5	= $ 12 000
Reinf. concrete headwalls = 2 @ $8 000 ea.	= $ 16 000
Supply & place backfill @	$ 5 000
Granular & asphalt restoration @	$ 5 000
Install precast sectional units @	$ 25 000
Type 'A' riprap as erosion protection @ channel bends = 300 m^3 @ 2.2t/m^3 = 660t @ $8/t	= $ 5 300
Guide rail = 250 m @ (8+20) = 250 m @ $28/m	= $ 7 000
&	
Guide posts = 70 @ (24+16) = 70 @ $40 ea.	= $ 2 800
:. Est. cost of culvert crossing	= $264 100
	Say $264 000

d) Erosion protection on the outfall portion
i.e. 125 m length @ approx. 3.2% gradient
:. 125x27x0.5 = 1 687.5 m^3 @ 2.2 t/m^3 = Say 3 700 t

:. 3 700 t @ $6/t	= $ 22 200
	Say $ 22 000

:. COMBINED ESTIMATED CONTRUCTION COST OF ROUTE 11 DIVERSION CHANNEL:

= $10 000 + $222 000 + $264 000 + $22 000 = $518 000

:. with a 20% allowance added for Engineering and Contingencies = $621 600

to which must be added an estimated land acquisition cost (north side, Sta. 1+300 to Sta. 1+700) of approx. 1.2 ha @ say $1 400/ha = $1 680, Say $ 1 700

For a Total Estimated Project Cost of $623 300.

9.0 BENEFIT-COST ANALYSES

9.1 GENERAL

The inclusion of a benefit-cost analysis permits the evaluation of the economic efficacy of a public investment in a capital project. In the case where there are several options available, the benefit-cost analysis provides an effective measure to identify the option providing the highest relative ratio of benefits to the cost of achieving those benefits.

As noted in the previous chapters, a series of remedial engineering works has been proposed to reduce the damages resulting from the flooding of Walker Brook. Each of these options has been costed and the consequential reduction in the forecast flood damages appraised. In addition, non-structural alternatives have been considered; but, given the difficulties of short-term forecasting for Walker Brook, it was recognized that such an alternative could only be designed to handle a flood which had already begun rather than reducing damages through the prediction of the event and the height of the floodwaters. Consequently, costs and benefits for the non-structural alternatives have not been considered.

It should be noted that where berms have been considered the costs include all construction, engineering, and contingency dollars to the top of the freeboard. On the other side of the equation, no benefits have been assigned for the protection/reduced damages resulting from the added freeboard allowance height of the berm.

A 20% allowance for engineering and contingencies has been allowed for in all structural modifications. For the calculation of the benefits, the non-site specific damages have been added to the site specific damages to provide the estimated reduction in damages arising from each structural change. (See Table 9-1.)

TABLE 9-1
ALLOCATION OF GENERAL AREA DAMAGES BY OPTION
(1988$)

Option		Campbellton	Atholville	NB DOT	NB Power	NB Tel	Personal	Commercial	Total
Sugarloaf	1:2	-	-	-	-	-	-	-	-
	1:20	-	-	-	-	-	-	-	-
	1:50	32 700	4 400	19 405	750	375	9 055	-	66 685
	1:100	56 400	5 300	23 580	1 000	500	9 055	-	95 835
Diversion	1:2	-	-	-	-	-	-	-	-
	1:20	80 200	-	11 270	8 140	4 070	24 915	7 250	135 845
	1:50	80 200	-	15 875	10 715	5 360	24 915	7 250	144 315
	1:100	80 200	-	19 210	12 875	6 440	24 915	7 250	150 890
Bridges - Rte. 11	1:2	-	-	-	-	-	-	-	-
	1:20	-	-	2 500	-	-	-	-	2 500
	1:50	-	-	3 500	-	-	-	-	3 500
	1:100	-	-	4 250	-	-	-	-	4 250
Upper Dover	1:2	-	-	-	-	-	-	-	-
	1:20	20 050	-	2 500	4 500	1 100	7 100	2 100	37 350
	1:50	20 050	-	3 500	5 500	1 500	7 100	2 100	39 750
	1:100	20 050	-	4 250	6 500	1 800	7 100	2 100	41 800
Alexander	1:2	-	-	-	-	-	-	-	-
	1:20	4 010	-	1 270	1 200	770	3 615	950	11 815
	1:50	4 010	-	1 875	1 215	860	3 615	950	12 525
	1:100	4 010	-	2 210	1 275	1 040	3 615	950	13 100
Pine	1:2	-	-	-	-	-	-	-	-
	1:20	28 070	-	2 500	1 220	1 100	7 100	2 100	42 090
	1:50	32 340(1)	-	3 500	2 000	1 500	7 100	2 100	48 540(1)
	1:100	33 710(2)	-	4 250	2 550	1 800	7 100	2 100	51 510(2)
Lower Dover	1:2	-	-	-	-	-	-	-	-
	1:20	28 070	-	2 500	1 200	1 100	7 100	2 100	42 070
	1:50	28 070	-	3 500	2 000	1 500	7 100	2 100	44 270
	1:100	28 070	-	4 250	2 550	1 800	7 100	2 100	45 870

(1) Includes $4 270 that is also included in the Mall Spill (See note Table 5-7)
(2) Includes $5 640 that is also included in the Mall Spill (See note Table 5-7)

Table 9-2, which follows, summarizes the specific area and general area damages by flood reduction option. The data from this table was then utilized to produce Table 9-3, Damages and Present Values by Flood Reduction Option.

The expected annual damage was calculated as defined in Section 5.4.1. The curves and calculations are provided in Figures 9-1 to 9-13 inclusive, which may be found at the end of this chapter.

The expected annual damage and the present values are summarized in Table 9-3.

9.2 ANALYSIS OF STRUCTURES

9.2.1 Sugarloaf Park Area Structures

The purpose of these works is to prevent spills from Walker Brook and a western tributary from flowing down the Val D'Amour Road Diversion into the Sugarloaf Mall parking lot and on down to the Val D'Amour Street/Alexander Street/Pine Street area.

The cost of the berms, floodwalls, and culvert on Route 270 is estimated at $80 400, and the land has been given a zero value since it is presently owned by the Province and is presently being used by the park and the Department of Transportation as a green belt, hence the use will not change.

The benefits, as shown in Table 9-3 as the present value of the Average Annual Cost of damages over 50 years, will provide benefit-cost ratios as follows:

- Benefits at 10% discount rate are $59 350, providing a benefit-cost ratio of 0.74:1.
- Benefits at 15% discount rate are $37 550, providing a benefit-cost ratio of 0.47:1.
- Benefits at 5% discount rate are $116 300, providing a benefit-cost ratio of 1.45:1.

TABLE 9-2
ALLOCATION OF DAMAGES BY FLOOD REDUCTION OPTION
(1988$)

		Return Period (years)			
Option		1:2	1:20	1:50	1:100
Val D'Amour	- Area Specific	$1 100	$2 600	$9 900	$16 000
	- General	-	5 010	7 055	8 540
	- Total	1 100	7 610	16 955	24,540
Mall Spill	- Area Specific	-	-	118 470	143 280
	- General	-	-	66 685	95 835
	- Total	-	-	185 155	239 115
Route 11 Diversion	- Area Specific	54 655	117 560	120 695	126 185
	- General	-	135 845	144 315	150 890
	- Total	54 655	253 405	265 010	277 075
Route 11 Culvert	- Area Specific	-	-	-	-
	- General	-	2 500	3 500	4 250
	- Total	-	2 500	3 500	4 250
Upper Dover St.	- Area Specific	48 375	79 820	81 605	81 880
	- General	-	37 350	39 750	41 800
	- Total	48 375	117 170	121 355	123 680
Alexander St.	- Area Specific	200	5 975	11 215	11 600
	- General	-	11 815	12 525	13 100
	- Total	200	17 790	23 740	24 700
Pine St.(1)	- Area Specific	1 000	12 150	13 710	15 800
	- General	-	42 090	48 540	51 510
	- Total	1 000	54 240	62 250	67 310
Pine St.(2)	- Area Specific	1 000	12 150	3 250	3 350
	- General	-	42 090	44 270	45 870
	- Total	1 000	54 240	47 520	49 220
Lower Dover St.	- Area Specific	5 080	19 615	24 625	29 355
	- General	-	42 070	44 270	45 870
	- Total	5 080	61 685	68 895	75 225

(1) Damage without occurrence of Mall Spill

(2) Damages reduced by amount common with Mall Spill

TABLE 9-3
DAMAGES & PRESENT VALUES BY FLOOD REDUCTION OPTION
(1988$)

Option	Flood Recurrence Damage 1:2	1:20	1:50	1:100	Annual Damage over 100 yrs.	% Annual Damage	Present Value of Annual Damage over 50 yrs. 10% Disc. Rate	15% Disc. Rate	5% Disc. Rate
Campbellton Area	54 655	253 405	450 165	516 190	123 990	100	1 110 150	702 400	2 174 500
Sugarloaf Structures	-	-	185 155	239 115	6 630	5.3	59 350	37 550	116 300
Rte. 11 Diversion(1)	54 655	253 405	265 010	277 075	117 360	94.7	1 050 800	664 850	2 058 250
Rte. 11 Culvert	-	2 500	3 500	4 250	1 170	0.9	10 500	6 650	20 500
Upper Dover Bridge	48 375	117 170	121 355	123 680	55 320	44.6	495 300	313 400	970 200
Alexander Bridge	200	17 790	23 740	24 700	8 210	6.6	73 500	46 500	144 000
Pine Bridge(1)	1 000	54 240	62 250	67 310	24 800	20.0	222 050	140 500	434 950
Pine Bridge(2)	1 000	54 240	54 240	54 240	24 460	19.7	219 000	138 550	429 000
Pine Dikes(1)	1 000	12 150	13 710	15 800	5 610	4.5	50 250	31 800	98 400
Pine Dikes(2)	1 000	12 150	12 150	12 150	5 520	4.5	49 400	31 250	96 800
Lr. Dover Bridge	5 080	61 685	68 895	75 225	28 350	22.8	253 850	160 600	497 200
Lr. Dover Dikes	5 080	19 615	24 625	29 335	9 320	7.5	83 450	52 800	163 450
Floodproofing	4 180	28 500	32 800	35 300	13 210	10.7	118 300	74 850	231 700
Val D'Amour Area	1 100	7 610	16 955	24 540	3 910	100	35 000	22 150	68 550
Val D'Amour Dike	-	-	6 050	12 500	310	7.9	2 800	1 750	5 450

(1) Damages including those that occur in common with the Mall Spill in 1:50 and 1:100 yr. recurrences.

(2) Damages do not include those that occur in common with the Mall Spill in 1:50 and 1:100 yr. recurrences.

9.2.2 Diversion Channel along Route 11

A diversion of Walker Brook floodwaters between the existing Route 11 crossing, immediately east of the Route 11/Route 270 interchange, and a point immediately downstream of Gerrard Street would eliminate flood flows through the two structures on Dover Street, and one each on Alexander and Pine Streets, thereby allowing these four structures to operate within their existing capacities, thus alleviating all damages due to backwater and spill resulting from the limited capacities of these four existing structures.

This option would eliminate Walker Brook floodwaters from the brook's existing course through a highly developed area of the City of Campbellton, and divert them to a new channel through a completely undeveloped area. The latter, from a planning point of view, would be very adequately separated from existing development by the Route 11 highway corridor.

As discussed in Chapter 7 previous, two alternatives for this option were assessed in detail, both on the south side of the existing Route 11. There was no serious consideration of developing the diversion on the north side of the existing two-lane Route 11 for two reasons: firstly, the proximity of such a development to the already existing development along the majority of this length, currently zoned as industrial (inside operations) and residential (single family, two family, and multiple, including the existing Sugarloaf Towers with 80 to 85 apartments); and secondly, since land would have to be acquired from these same private property owners for such a development on the north side, and this could prove to be quite difficult should there be opposition to the development from any landowners.

The channel location option recommended in Chapter 7, at an estimated cost of $623 300, has been used in this benefit-cost analysis.

The benefits, as shown in Table 9-3, will provide benefit-cost ratios as follows:

- Benefits at a 10% discount rate are $1 050 800, providing a benefit-cost ratio of 1.69:1.
- Benefits at a 15% discount rate area $664 850, providing a benefit-cost ratio of 1.07:1.
- Benefits at a 5% discount rate are $2 058 250, providing a benefit-cost ratio of 3.30:1.

9.2.3 Sugarloaf Structures plus Route 11 Diversion Channel

The combination of the roadside structures adjacent Sugarloaf Provincial Park together with the Route 11 diversion channel would effectively remove all flood damage risk to the "St. Albert Area" of Campbellton/ Atholville arising from Walker Brook and its major tributaries.

The combined estimated cost of construction of the required structures is $703 700.

The benefits, shown in Table 9-3, will provide benefit-cost ratios as follows:

- Benefits at a 10% discount rate are $1 110 150, providing a benefit-cost ratio of 1.58:1.
- Benefits at a 15% discount rate are $702 400, providing a benefit-cost ratio of 1.00:1.
- Benefits at a 5% percent discount rate are $2 174 500, providing a benefit-cost ratio of 3.09:1.

9.2.4 Bridge Structures

To provide remedial action equivalent to the diversion along Route

11, five crossings would have to be upgraded to accept flood flows without creating the backwaters or spills that cause property damage. The structures will first be considered individually, and then in total.

Lower Dover Street Crossing

A bridge is proposed for the lower Dover Street crossing of Walker Brook to replace the existing culvert.

The cost of removing the existing culvert and replacing it with a bridge has been estimated at $738 000.

The benefits arising from this structure are equal to the total annual damage to adjacent properties for 50 years reduced to the present value. The benefits, as shown in Table 9-3, will provide benefit-cost ratios as follows:

- Benefits at a 10% discount rate are $253 850, providing a benefit cost ratio of 0.34:1.
- Benefits at a 15% discount rate are $160 600, providing a benefit-cost ratio of 0.22:1.
- Benefits at a 5% discount rate are $497 200, providing a benefit-cost ratio of 0.67:1.

Pine Street Crossing

To eliminate flood backwater damage caused by an inadequate culvert at Pine Street, a bridge structure is proposed.

The cost of the removal of the culvert and fill and the construction of a bridge has been estimated at $374 400.

The benefits, shown in Table 9-3, are independent of those arising from the Mall Spill and will provide benefit-cost ratios as follows:

- Benefits at a 10% discount rate are $222 050, providing a benefit-cost ratio of 0.59:1.
- Bendfits at a 15% discount rate are $140 500, providing a benefit-cost ratio of 0.38:1.
- Benefits at a 5% discount rate are $434 950, providing a benefit-cost ratio of 1.16:1.

Alexander Street Crossing

The constricted flow through the Alexander Street footbridge results in a backwater and spill at flood flows. It has therefore been proposed to double the span of the footbridge to allow full passage of the floodwaters beneath the structure.

The cost of the new span has been estimated at $38 400.

The present values of the 50 year reduction in flood damage are shown in Table 9-3. They will provide benefit-cost ratios as follows:

- Benefits at a 10% discount rate amount to $73 500, providing a benefit-cost ratio of 1.91:1.
- Benefits at a 15% discount rate amount to $46 500, providing a benefit-cost ratio of 1.21:1.
- Benefits at a 5% discount rate amount to $144 000, providing a benefit-cost ratio of 3.75:1.

Upper Dover Street Crossing

Full flow of 1:20 year floodwaters, and greater, are severely restricted by the upper Dover Street culvert, causing a considerable backwater and spill over Dover Street north of the crossing. To remedy the problem a bridge has been proposed.

The cost of removing the existing structure and replacing it with a bridge has been estimated to be $670 800.

The benefits arising from the new structure are shown in Table 9-3. The benefit-cost ratios arising therefrom are as follows:

- Benefits at 10% discount rate are $495 300, providing a benefit-cost ratio of 0.74:1.
- Benefits at 15% discount rate are $313 400, providing a benefit-cost ratio of 0.47:1.
- Benefits at 5% discount rate are $970 200, providing a benefit-cost ratio of 1.45:1.

Route 11 Crossing

The undersized culvert at the existing Route 11 crossing of Walker Brook results in a spill flowing down the ditch, eastward, on the south side of the highway. Some of this spill floods the highway at a point 900 metres east of the culvert while the rest utilizes an existing culvert to rejoin the Brook below Gerrard Street.

The cost of increasing the capacity of the Route 11 crossing to accommodate the full flood flows of Walker Brook has been estimated at $120 000, based on providing an additional culvert.

The benefits arising from the new Route 11 culvert-arch structure are provided in Table 9-3. The benefit-cost ratios are as follows:

- Benefits at a 10% discount rate are $10 500, providing a benefit-cost ratio of 0.09:1.

- Benefits at a 15% discount rate are $6 650, providing a benefit-cost ratio of 0.06:1.
- Benefits at a 5% discount rate are $20 500, providing a benefit-cost ratio of 0.17:1.

Combination of All Five Crossings

The construction of all five new structures over Walker Brook from the Lower Dover Street crossing up to and including the Route 11 crossing will eliminate all backwaters and spills caused by the existing structures.

The total cost of the installation of the new structures has been estimated at $1 941 600.

The present value of the benefits arising from the construction of the five new crossings is the sum of the individual benefits shown for each structure/option in Table 9-3. The benefit-cost ratios are as follows:

- Benefits calculated at a 10% discount rate total $1 053 800, and create a benefit-cost ratio of 0.54:1.
- Benefits calculated at a 15% discount rate total $666 800, and have a benefit-cost ratio of 0.34:1.
- Benefits calculated at a 5% discount rate total $2 064 200, and have a benefit-cost ratio of 1.06:1.

9.2.5 Crossing Structures plus Sugarloaf Structures

The construction of the berms adjacent Sugarloaf Park in addition to the five new stream crossing structures identified above will eliminate all flooding in the St. Albert area of Campbellton and Atholville.

The total cost of the removal of existing inadequate structures and the construction of the new structures has been estimated at $2 022 000.

The present value of the benefits arising from this combination are the sum of the individual benefits for each option less those benefits that arise jointly from the Mall Spill and the Pine Street backwater. In Table 9-3 this is provided by the Pine Street option denoted by footnote [2]. The total benefits and the resultant benefit-costs are as follows:

- Benefits calculated at a 10% discount rate are $1 110 150, which provides a benefit-cost ratio of 0.55:1.
- Benefits calculated at a 15% discount rate are $702 400, which provides a benefit-cost ratio of 0.35:1.
- Benefits calculated at a 5% discount rate are $2 174 500, which provides a benefit-cost ratio of 1.08:1.

9.2.6 Dikes Adjacent Crossings

Dikes have been proposed to provide property protection adjacent two crossings within the City of Campbellton: at Lower Dover Street and at Pine/Alexander Streets.

Lower Dover Street Dikes

Dikes are proposed on Dover Street east and west of Ortona and along the east side of Ortona Street. The cost of constructing these berms has been estimated at $78 000.

The present values of the average annual benefits over the 50 year term are provided in Table 9-3. The resultant benefit-cost ratios for the three discount rates are as follows:

- Benefits accruing with a 10% discount rate are $83 450, which results in a benefit-cost ratio of 1.07:1.
- Benefits accruing with a 15% discount rate are $52 800, which results in a benefit-cost ratio of 0.68:1.
- Benefits accruing with a 5% discount rate are $163 450, which results in a benefit-cost ratio of 2.10:1.

Pine/Alexander Streets Dikes

Dikes proposed on both sides of the Brook have been estimated to cost a total of $63 600.

The present values of the average annual benefits over the 50 year term are provided in Table 9-3. The benefit-cost ratios that result are provided below:

- Benefits obtained with a 10% discount rate are $50 250, and create a benefit-cost ratio of 0.79:1.
- Benefits obtained with a 15% discount rate are $31 800, and create a benefit-cost ratio of 0.50:1.
- Benefits obtained with a 5% discount rate are $98 400, and create a benefit-cost ratio of 1.55:1.

9.2.7 Structures at Sugarloaf and Dikes at Crossings

Utilizing dikes, floodwalls and a flood gate near Sugarloaf Park as well as remedial dikes at the lower Dover Street crossing and in the Alexander/Pine Street Area has the potential to reduce damage by the flooding of Walker Brook by 30%.

The total estimated cost of the required structures is comparatively low, at $222 000.

The benefits are slightly less than the sum of the individual benefits because of the Pine Street backwater and the Mall Spill affecting the same area on Val D'Amour Street (near the corner of Pine Street). The benefits and the resulting benefit-cost ratios are provided below:

- Benefits at the 10% discount rate are $192 200, producing a benefit-cost ratio of 0.87:1.
- Benefits at the 15% discount rate are $121 600, producing a benefit-cost ratio of 0.55:1.
- Benefits at the 5% discount rate are $376 550, producing a benefit-cost ratio of 1.70:1.

9.2.8 Floodproofing

Two methods of floodproofing were suggested in Section 6.3.2. The first consists of the raising of buildings above the floodwaters; the second requires the waterproofing of basement walls and other flood entry points.

The first option would appear to be relevant for only one building, a single storey bungalow without a basement, located at 44 Alexander Street.

The second option could provide protection to seven houses at the lower Dover Street crossing area and two on Val D'Amour Street near Pine Street.

Although this option could prevent damage to the ten structures it would not prevent serious erosion to four landscaped properties, clean-up costs, and fence repairs that have been estimated as follows in lieu of damages as calculated by the Paragon Model.

Flood Return Period	Savings
1:2 year	4 180
1:20 year	28 500
1:50 year	32 800
1:100 year	35 300

The costs of the floodproofing have been estimated at $141 600.

The present value of the benefits, as provided in Table 9-3, are given below together with the resultant benefit-cost ratio for each discount rate:

- Benefits calculated at a 10% discount rate are $118 300, which create a benefit-cost ratio of 0.84:1.
- Benefits calculated at a 15% discount rate are $74 850, creating a benefit-cost ratio of 0.53:1.
- Benefits calculated at a 5% discount rate are $231 700, creating a benefit-cost ratio of 1.64:1.

9.2.9 Floodproofing plus Sugarloaf Structures

The combination of floodproofing and the construction of the Sugarloaf structures will provide an overall reduction of 30% in the average annual flood damage.

The cost of this option is also estimated to be $222 000 (i.e. equivalent to that of Section 9.2.7 previous).

- Benefits at a 10% discount rate will total $177 650, providing a benefit-cost ratio of 0.80:1.
- Benefits at a 15% discount rate will be $112 400, providing a benefit-cost ratio of 0.51:1.
- Benefits at a 5% discount rate will be $348 000, providing a benefit-cost ratio of 1.57:1.

9.2.10 Val D'Amour Community

In the community of Val D'Amour the hydraulic analysis has identified only one building within the 1:50 year return period inundation. A small dike will protect this building, a service station.

The cost of the dike has been estimated at $6 000.

The present value of the benefits arising from the protection of the service station over a 50 year period at the three discount rates is provided in Table 9-3. The resultant benefit-cost ratios are as follows:

- Benefits calculated at a 10% discount rate are $2 800, creating a benefit-cost ratio of 0.47:1.
- Benefits calculated at a 15% discount rate are $1 750, creating a benefit-cost ratio of 0.29:1.
- Benefits calculated at a 5% discount rate are $5 450, creating a benefit-cost ratio of 0.91:1.

9.2.11 Channel Improvements

Channel improvements had been suggested for consideration for the stream between Gerrard Street and Gallant Drive, and for the stretch downstream of Village Avenue, including the removal of the ruins of the old dam. The hydraulic analysis for these two areas did not indicate potential damages to structures along these reaches. Consequently, since there are no benefits to be accrued from channel improvements, a benefit-cost ratio cannot be calculated.

9.3 BENEFIT-COST EVALUATION

In order to establish a ranking of the options, a simple formula has been employed. The reason for its use is to recognize the combined importance of the degree of reduction of Average Annual Damage as well as the Benefit-Cost ratio for each Flood Reduction Option. The formula provides for the multiplication of the Benefit-Cost ratio, established for the 10% discount rate, by the percent of reduction in the Average Annual Damage for each specific option. The results, in hierarchical order, are provided in Table 9-4, which follows.

9.3.1 Campbellton Area

For the purposes of this section the "Campbellton Area" is considered as being all that portion of the Walker Brook flood plain that lies between the entrance to Sugarloaf Provincial Park and the Restigouche River.

A quick review of the remedial options clearly focuses attention on the Route 11 Diversion Channel. Not only does it provide the optimum benefit-cost ratio, it also provides a high degree of protection for all levels of flooding.

The remedial action adjacent the entrance of Sugarloaf Park by itself only reduces the estimated average annual damage by 5.3%. This is because the damage occurs only at the 1:50 year and above recurrence intervals; however, when this spill occurs the damage is considerable. The combination of construction of the Route 11 Diversion Channel together with the structures to prevent the Mall Spill will effectively eliminate the flood hazard of the Walker Brook in the Campbellton area. Combined, they provide a positive benefit-cost ratio of 1.58:1.

Two other individual options also have positive benefit-cost ratios. The Alexander Street footbridge provides a ratio of 1.91:1, but only reduces the estimated average annual damage by 6.6%.

TABLE 9-4
HIERARCHIAL RATING OF FLOOD DAMAGE REDUCTION OPTIONS

Area/Option	Cost (1988$)	% Average Annual Damage Reduction(ADR)	Benefit/Cost Ratio @ 10% Discount	% ADR x B/C Ratio
Campbellton Area:				
Route 11 Diversion Channel	623 300	94.7	1.69:1	160.0
Route 11 Diversion Channel & Sugarloaf Structures	703 700	100.0	1.58:1	158.0
4 Bridges, Rte. 11 Culvert & Sugarloaf Structures	2 022 000	100.0	0.55:1	55.0
4 Bridges & Rte. 11 Culvert	1 941 600	94.7	0.54:1	51.1
Upper Dover Street Bridge	670 800	44.6	0.74:1	33.0
Sugarloaf Structures & Dikes at Alexander, Pine & Lower Dover Streets	222 000	17.3	0.87:1	15.1
Sugarloaf Structures & Floodproofing	222 000	16.0	0.80:1	12.8
Alexander Street Footbridge	38 400	6.6	1.91:1	12.6
Pine Street Bridge	374 400	20.0	0.59:1	11.8
Floodproofing at Alexander, Pine & Lower Dover Streets	141 600	10.7	0.84:1	9.0
Lower Dover Street Dikes	78 000	7.5	1.07:1	8.0
Lower Dover Street Bridge	738 000	22.8	0.34:1	7.8
Sugarloaf Structures	80 400	5.3	0.73:1	3.9
Pine & Alexander Street Dikes	63 600	4.5	0.79:1	3.6
Route 11 Culvert	120 000	0.9	0.09:1	0.1
Val D'Amour Area:				
Service Station Dike	6 000	7.9	0.47:1	3.7

The lower Dover Street bridge is also positive, at 1.07:1; but, with a cost in excess of the combination of the Route 11 Channel Diversion and the Sugarloaf Structures, it only provides 23% of the protection the other two afford.

In conclusion, the option of constructing the diversion channel alongside Route 11 provides the greatest level and percentage of benefits for the least cost. When combined with the structures at the entrance to Sugarloaf Park, all damages can be eliminated with only a minor reduction in benefit-cost ratio. The options are discussed in Table 9-5, which follows.

TABLE 9-5
CAMPBELLTON AREA REMEDIAL FLOOD DAMAGE REDUCTION OPTIONS

Remedial Option	Data	Comments
Route 11 Diversion Channel Construction of a diversion channel on south side of Route 11. Includes structure to divert Brook into Channel at 1:2 year and greater flows.	Cost - $623 300 EAADR (1) - 94.7% B/C ratio 1.69:1	- Will eliminate flooding except the Mall Spill, which only occurs at, or approaching, the 1:50 year flood flows. - Maintenance costs should be minimal as channel gradients should inhibit siltation. - Will allow normal Walker Brook flow through Campbellton except during flood stage, therefore no damage to aquatic habitat.
Sugarloaf Structures Construction of dikes, floodwall and enlarged culvert near Sugarloaf Park entrance plus protection from tributary run-off.	Cost - $80 400 EAADR - 5.3% B/C ratio 0.73:1	- Will prevent the Mall Spill from causing damages at or near 1:50 year flood flows. - Once grassed, the structures should require virtually no maintenance unless broached. - Although EAADR is low, the damages when they occur are high, but at low frequency.

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1. EAADR - Estimated Average Annual Damage Reduction

TABLE 9-5
CAMPBELLTON AREA REMEDIAL FLOOD REDUCTION OPTION (cont'd)

Remedial Option	Data	Comments
Route 11 Diversion Channel & Sugarloaf Structures The combination of the two foregoing options	Cost - $703 700 EAADR - 100.0% B/C ratio 1.58.1	- Will prevent Walker Brook from causing flood damage in Campbellton Area. - The level of protection is excellent when related to the benefits and the costs. - Maintenance costs will be minimal.
Alexander Street Footbridge Construction of a span twice the length of the existing bridge.	Cost - $38 400 EAADR - 6.6% B/C ratio 1.91:1	- Excellent benefit-cost ratio but has little impact on damage reduction. - Has to be combined with the other 3 bridges to be of any major impact.
4 Bridges & Route 11 Culvert Construction of bridges to replace existing structures at Pine and Alexander Streets and the upper and lower crossings of Dover Street plus the twinning of the Route 11 culvert.	Cost - $1 941 600 EAADR - 94.7% B/C ratio 0.54.1	- Will provide the same degree of flood reduction as the Diversion Channel but at much higher cost (more than 3 times). - Can also be combined with Sugarloaf Structures but high costs remain the problem.
Diking Construction of protective dikes near the Alexander, Pine, and Lower Dover Street crossings.	Cost - $141 600 EAADR - 12.0% B/C ratio 0.94.1	- Although the benefit-cost ratio is comparatively high the costs do achieve a 12% reduction in the estimated average annual damages. - May cause problems with drainage at individual houses, grades on driveways, and with landscaping. - Will require easements on private property to construct dikes.

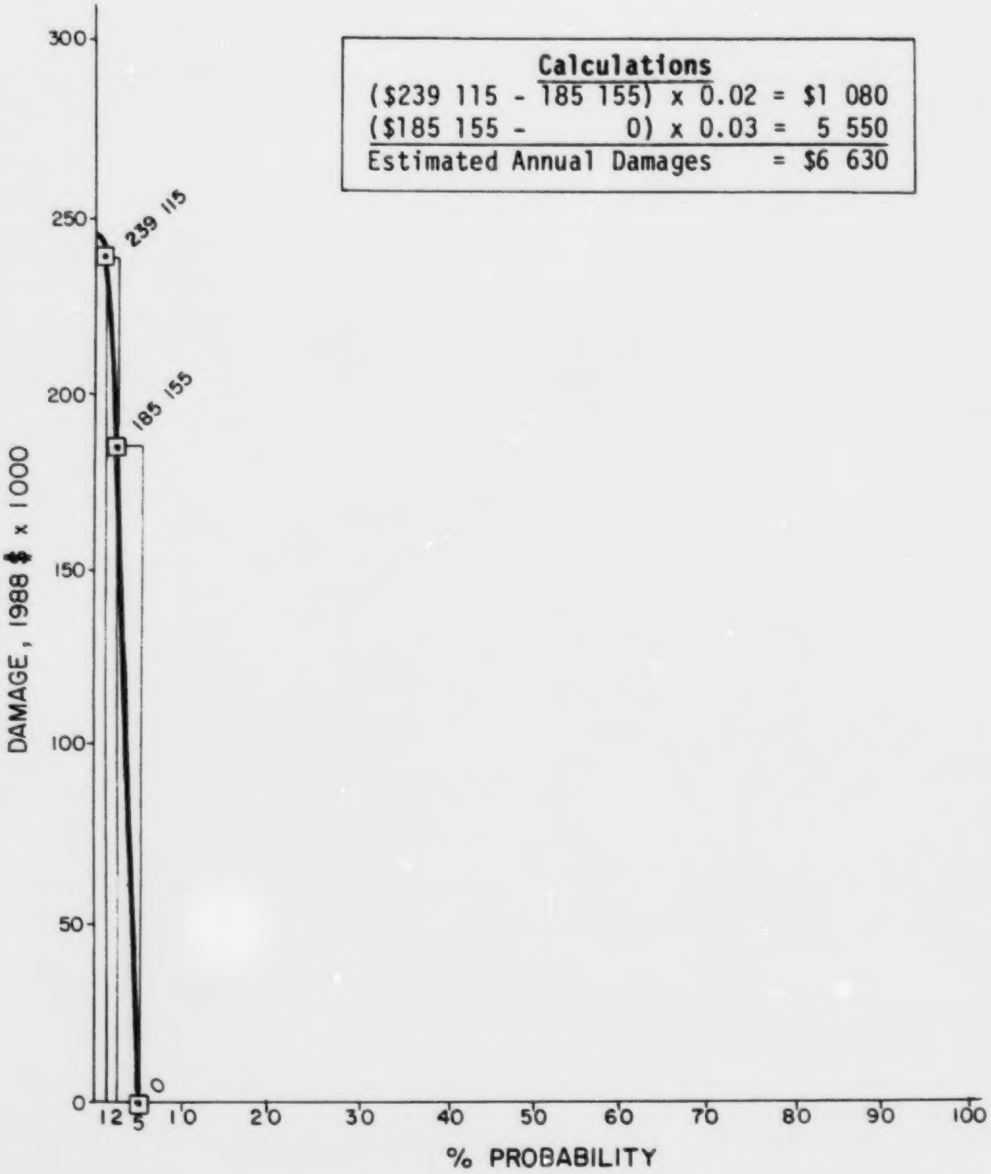
9.3.2 Val D'Amour Area

In the Val D'Amour community only one structure was identified as being prone to flood damage. In addition to the service station so affected, another commercial building may have access reduced.

TABLE 9-6
VAL D'AMOUR AREA REMEDIAL
FLOOD DAMAGE REDUCTION OPTIONS

Remedial Option	Data	Comments
Berm at Service Station		
Construction of a berm to redirect bank overflow back into stream behind station	Cost - $6 000 EAADR - 7.9% B/C ratio - 0.47:1	This will prevent the incursion of the 1:50 and the 1:100 year floods into the service station, but will not reduce damages to public property nor to utilities.

It would appear that the benefits accruing from this option are too low to be entertained.



Calculations

($239 115 - 185 155) x 0.02 = $1 080

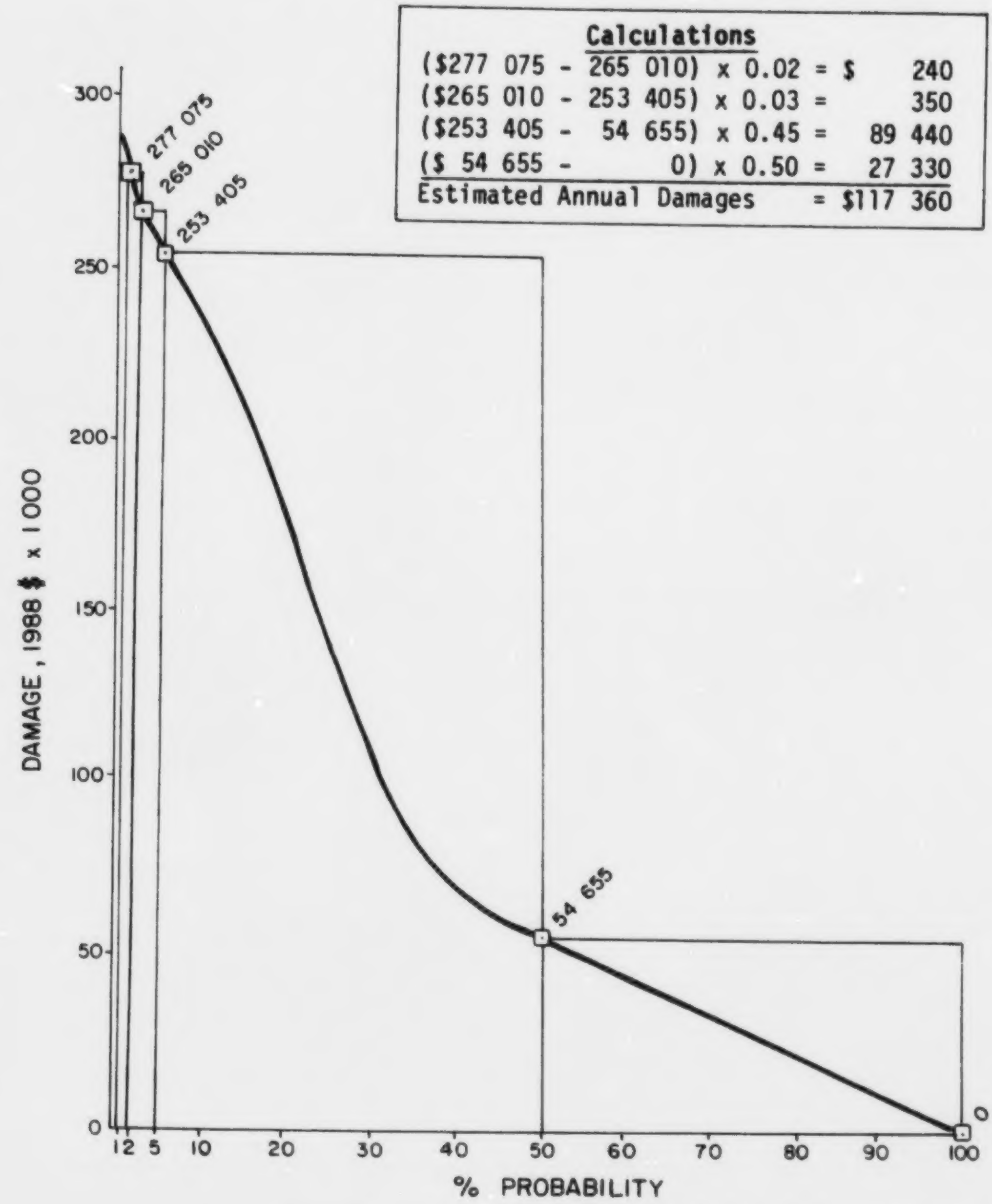
($185 155 - 0) x 0.03 = 5 550

Estimated Annual Damages = $6 630

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Mall Spill
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-1



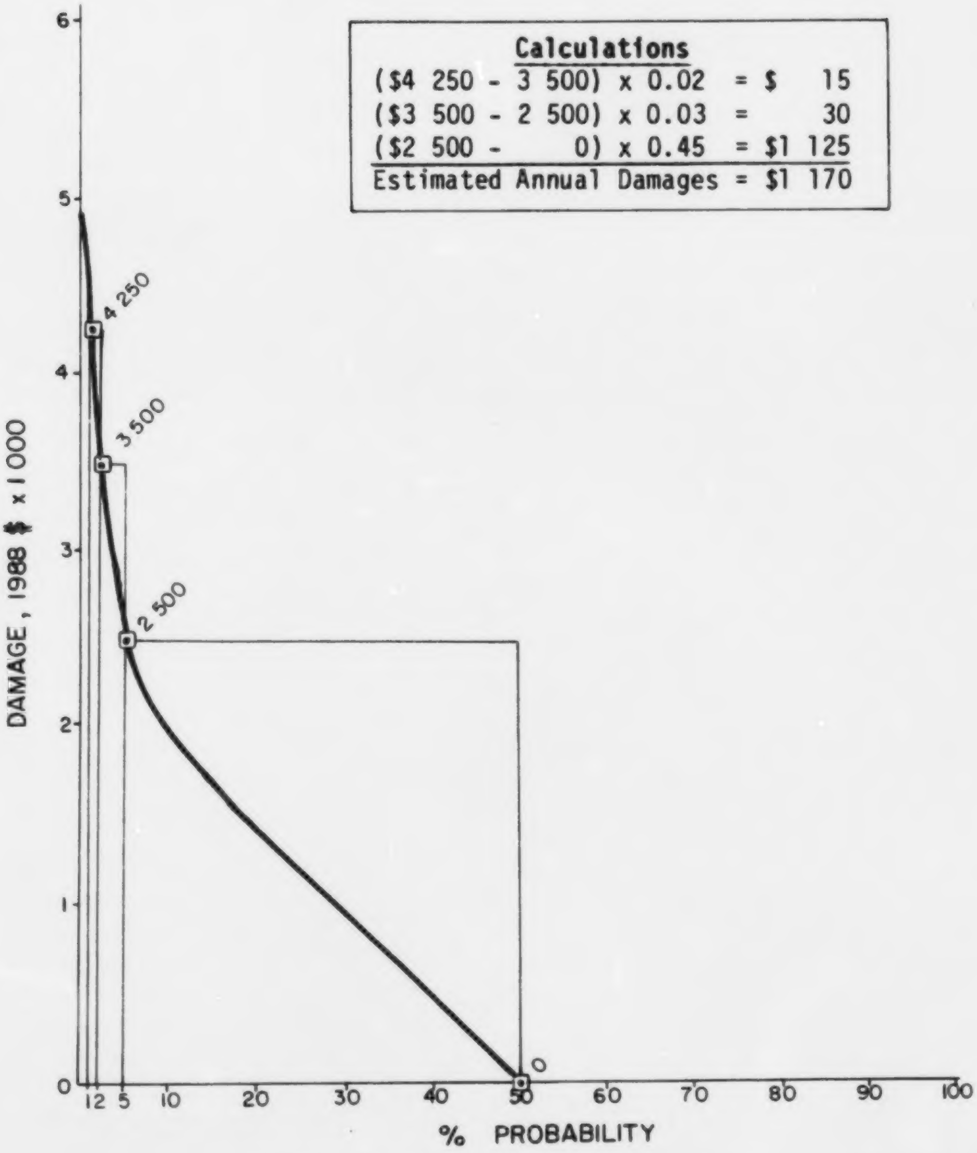
Calculations

($277 075 - 265 010) x 0.02 = $ 240
($265 010 - 253 405) x 0.03 = 350
($253 405 - 54 655) x 0.45 = 89 440
($ 54 655 - 0) x 0.50 = 27 330
Estimated Annual Damages = $117 360

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Route 11 Diversion Channel
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-2



Calculations

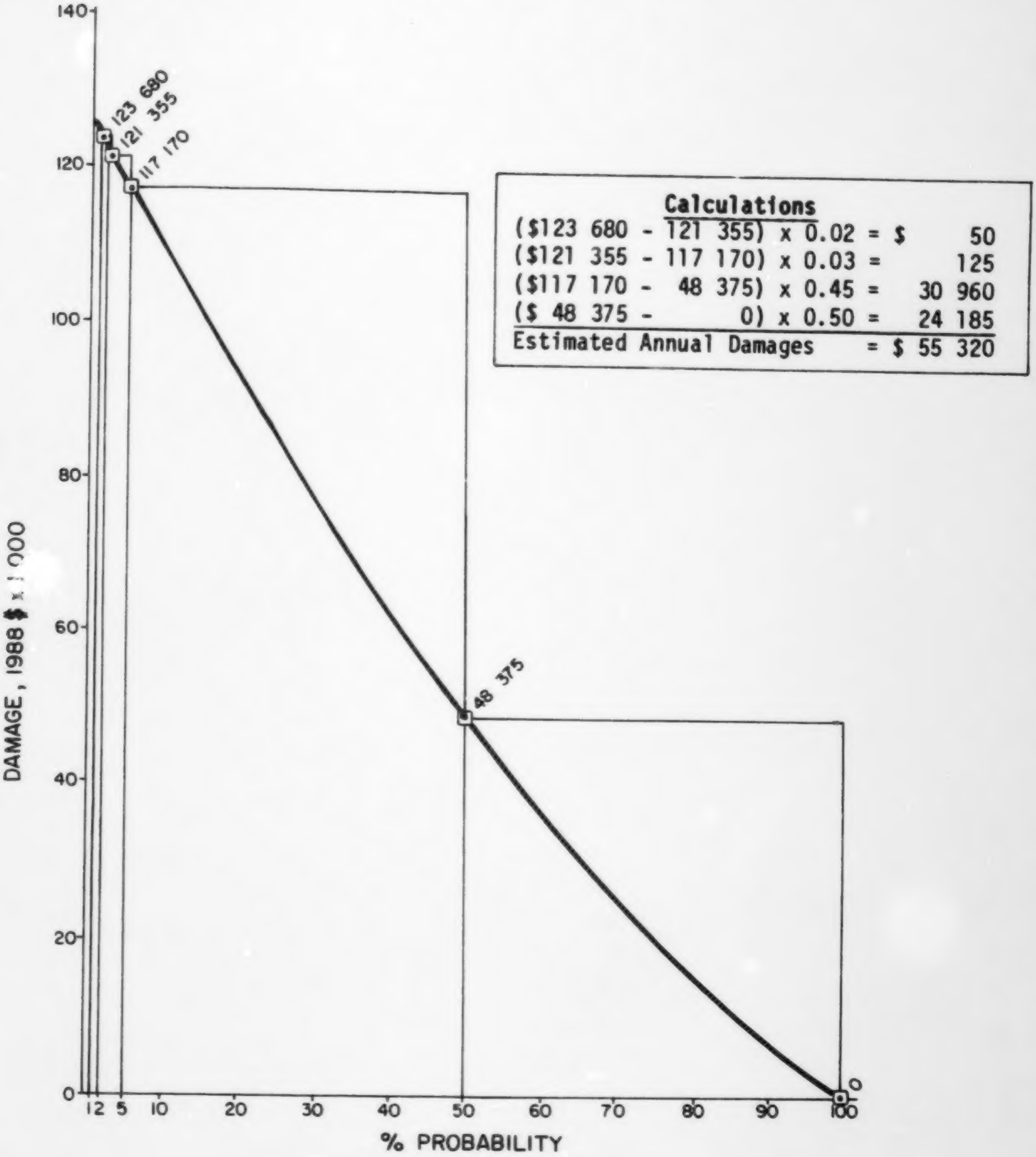
($4 250 - 3 500) x 0.02 = $ 15
($3 500 - 2 500) x 0.03 = 30
($2 500 - 0) x 0.45 = $1 125

Estimated Annual Damages = $1 170

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Route 11 Culvert
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-3



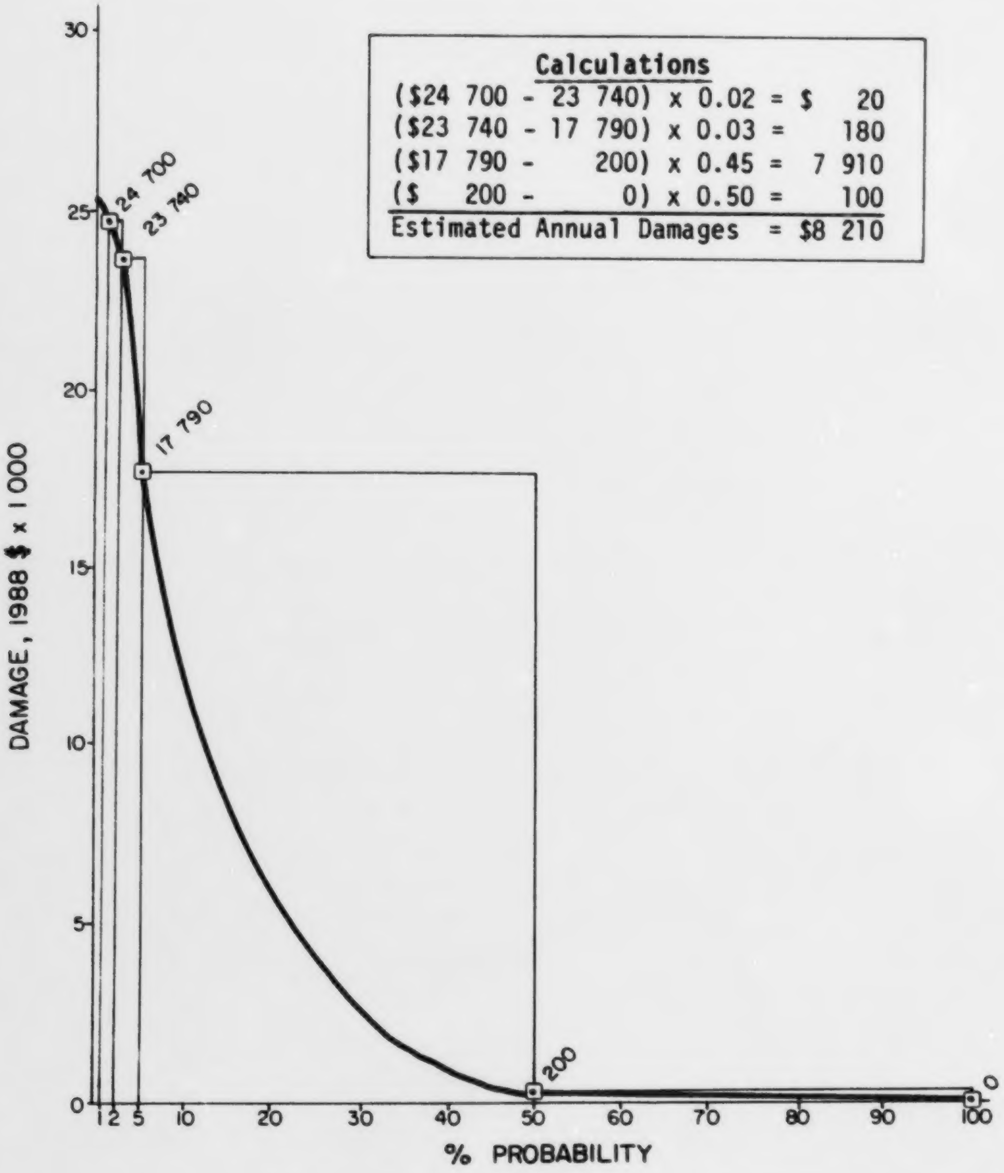
Calculations

(\$123 680 - 121 355) x 0.02 = \$ 50
(\$121 355 - 117 170) x 0.03 = 125
(\$117 170 - 48 375) x 0.45 = 30 960
(\$ 48 375 - 0) x 0.50 = 24 185
Estimated Annual Damages = \$ 55 320

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Upper Dover Street Bridge
Estimated Annual Damages Curve and Calculations
(1988 \$)

FENCO FIG. 9-4



Calculations

($24 700 - 23 740) x 0.02 = $ 20

($23 740 - 17 790) x 0.03 = 180

($17 790 - 200) x 0.45 = 7 910

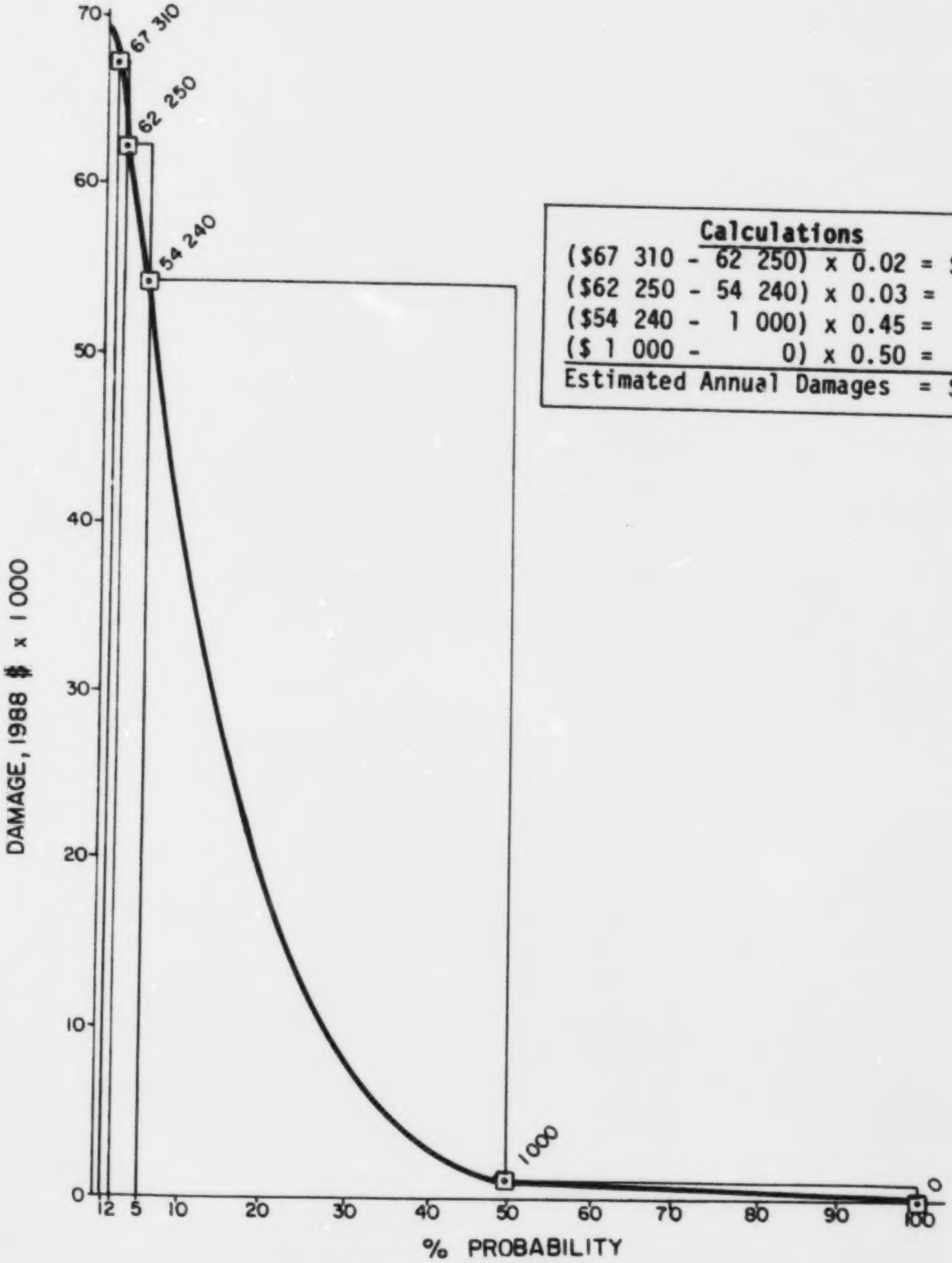
($ 200 - 0) x 0.50 = 100

Estimated Annual Damages = $8 210

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Alexander Street Bridge
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-5



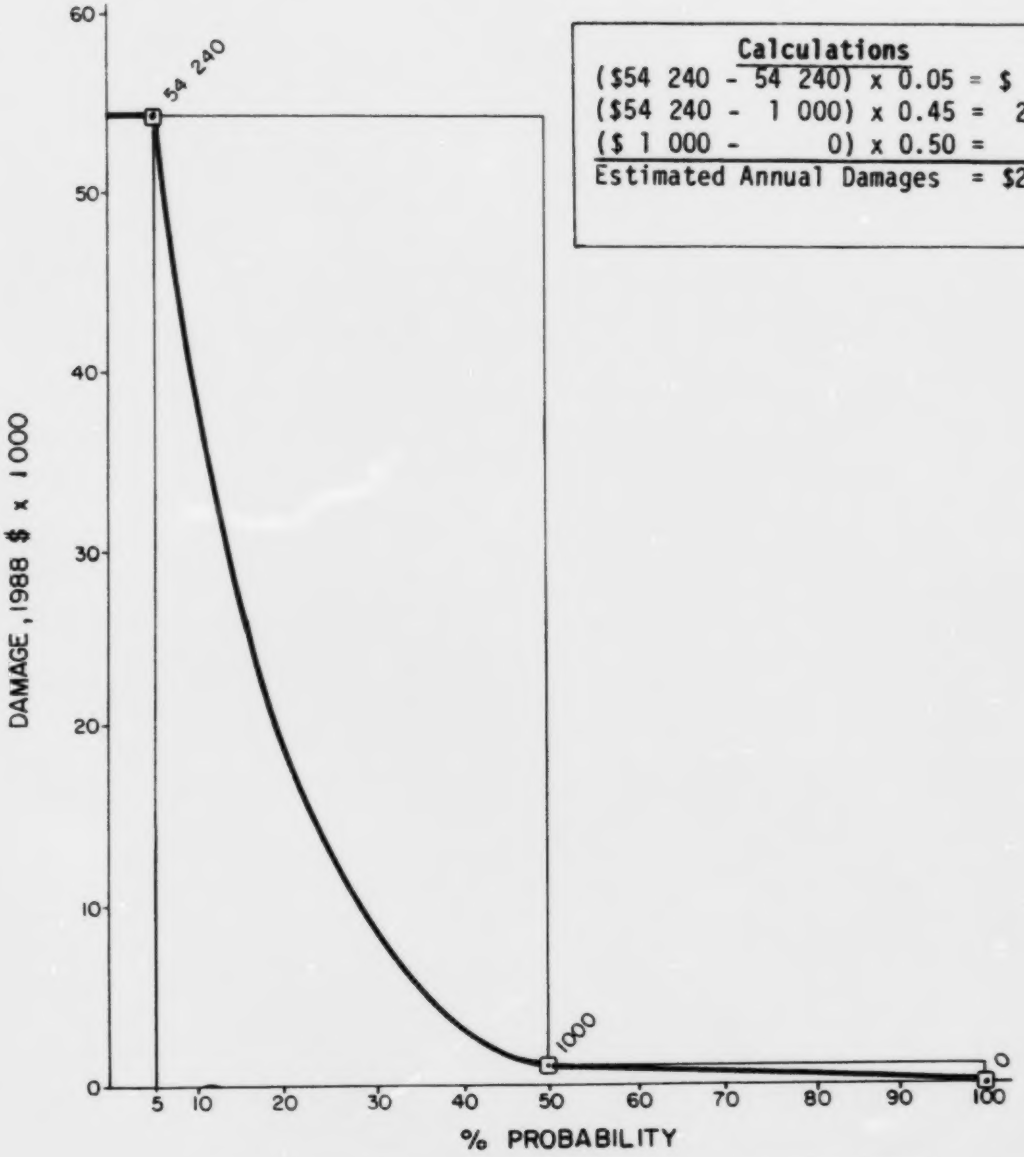
Calculations

($67 310 - 62 250) x 0.02 = $ 100
($62 250 - 54 240) x 0.03 = 240
($54 240 - 1 000) x 0.45 = 23 960
($ 1 000 - 0) x 0.50 = 500
Estimated Annual Damages = $24 800

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Pine Street Bridge
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-6



Calculations

($54 240 - 54 240) x 0.05 = $ 0

($54 240 - 1 000) x 0.45 = 23 960

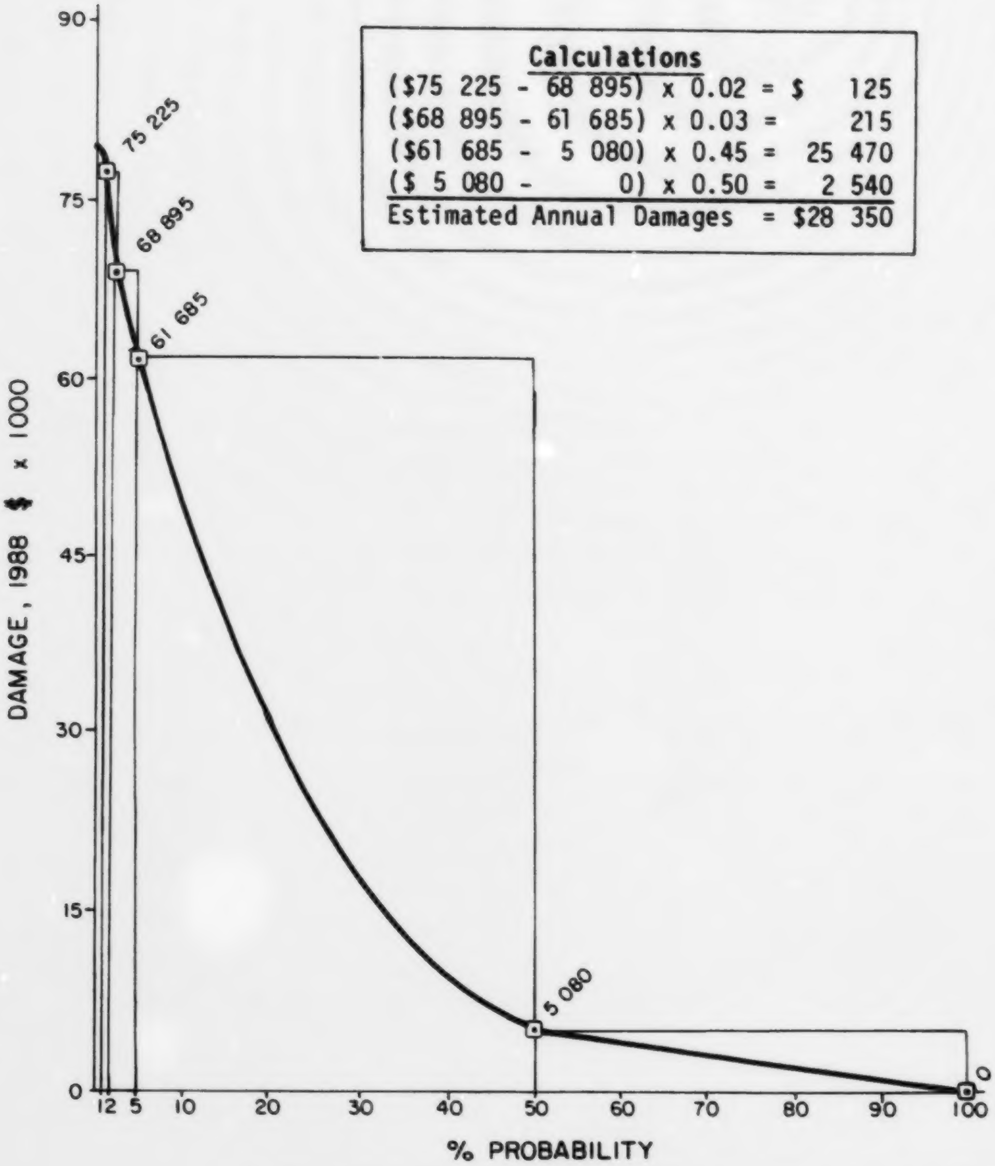
($ 1 000 - 0) x 0.50 = 500

Estimated Annual Damages = $24 460

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Pine St. Bridge reduced by Mall Spill
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-7



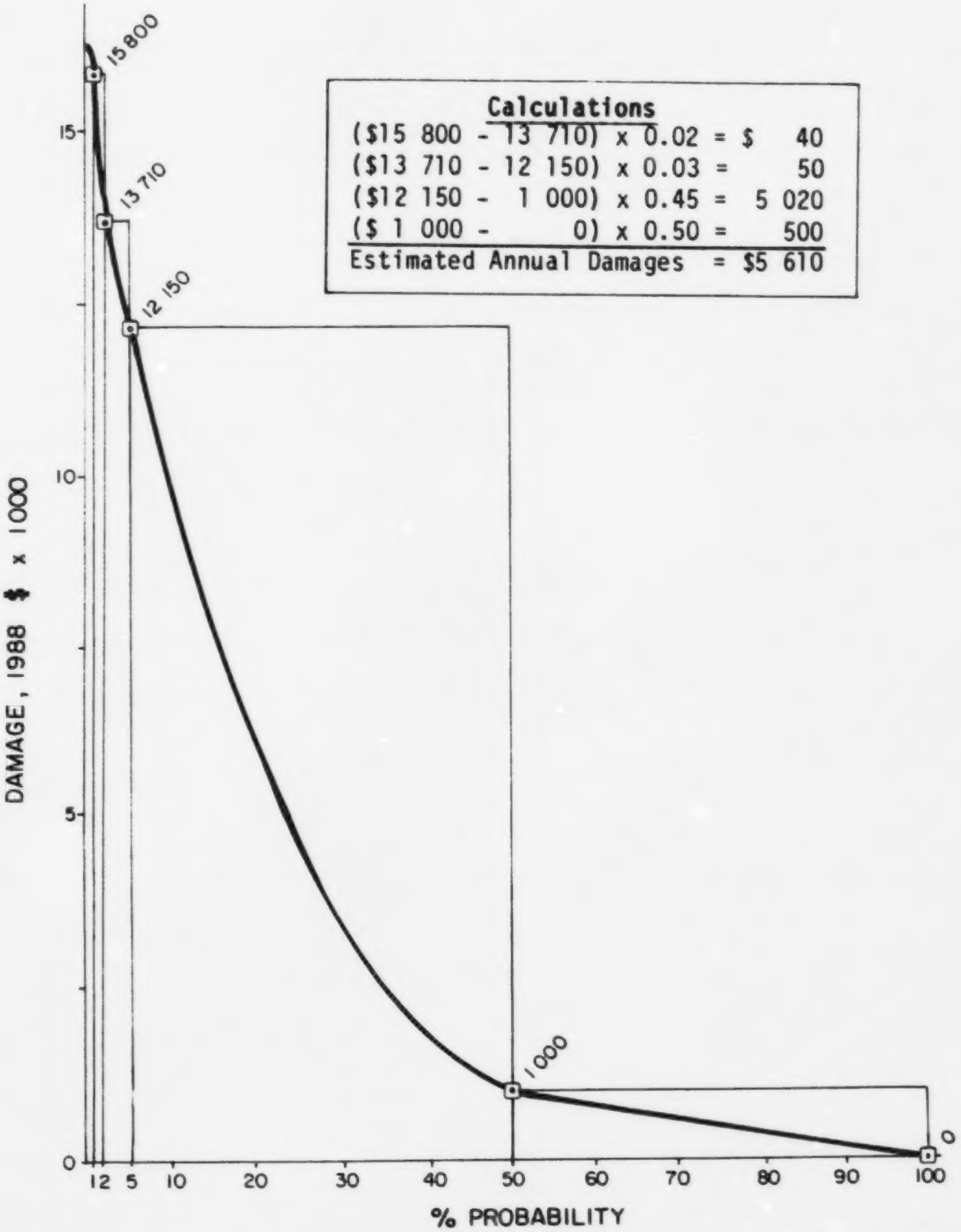
Calculations

($75 225 - 68 895) x 0.02 = $ 125
($68 895 - 61 685) x 0.03 = 215
($61 685 - 5 080) x 0.45 = 25 470
($ 5 080 - 0) x 0.50 = 2 540
Estimated Annual Damages = $28 350

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Lower Dover Street Bridge
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-8



Calculations

($15 800 - 13 710) x 0.02 = $ 40

($13 710 - 12 150) x 0.03 = 50

($12 150 - 1 000) x 0.45 = 5 020

($ 1 000 - 0) x 0.50 = 500

Estimated Annual Damages = $5 610

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Pine Street Dikes
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-9

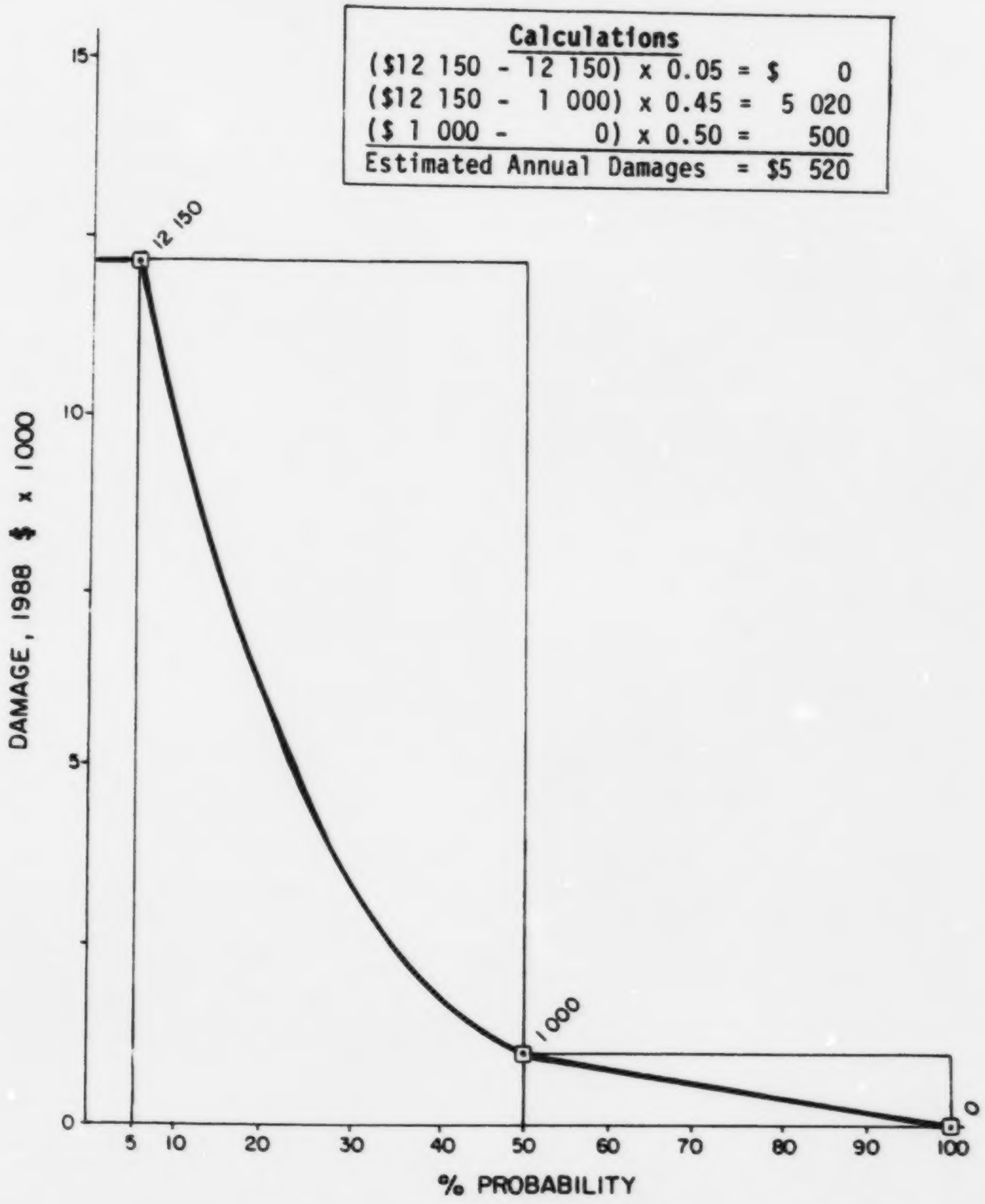
Calculations

(\$12 150 - 12 150) x 0.05 = \$ 0

(\$12 150 - 1 000) x 0.45 = 5 020

(\$ 1 000 - 0) x 0.50 = 500

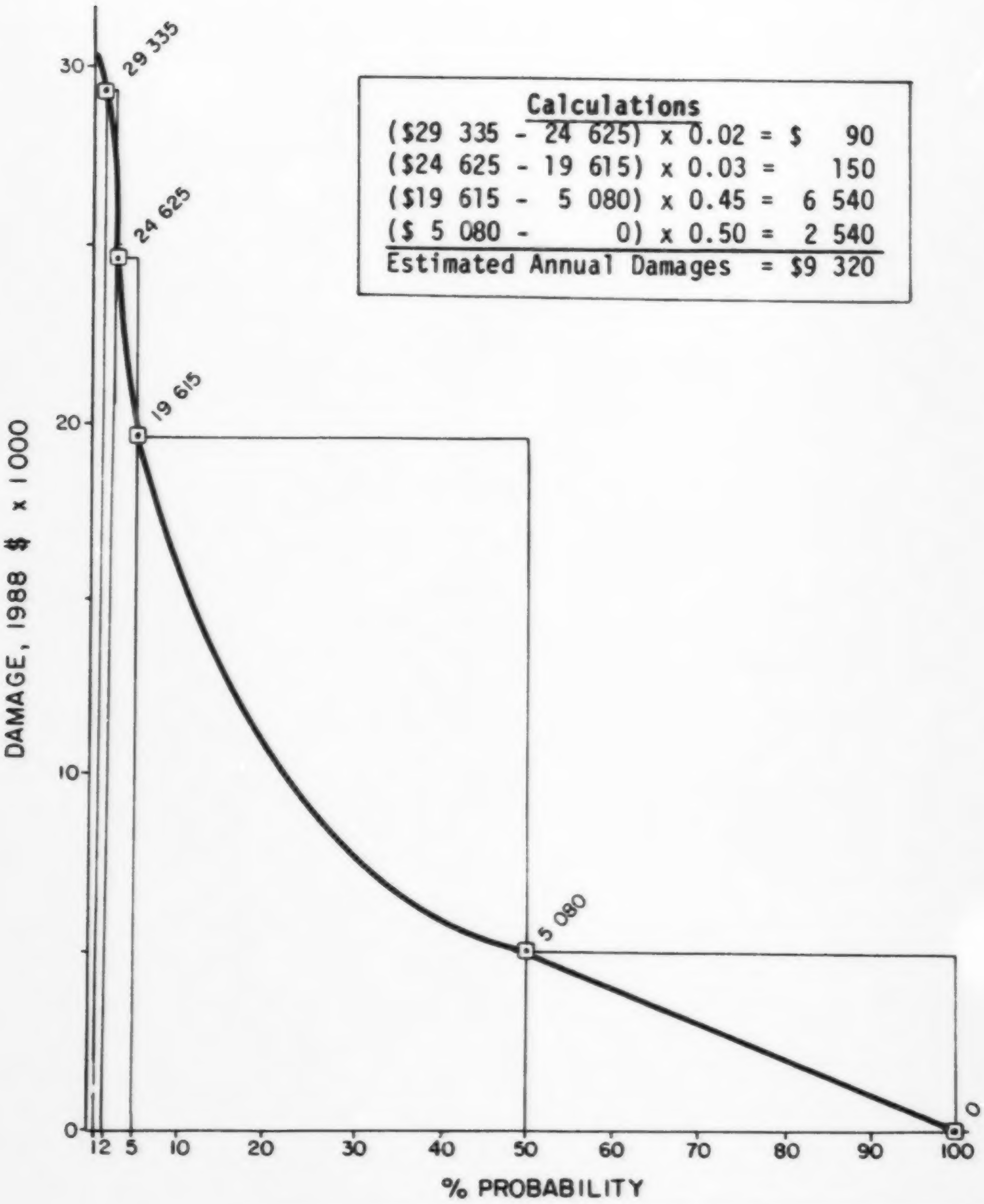
Estimated Annual Damages = \$5 520



WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Pine St. Dikes reduced by Mall Spill
Estimated Annual Damages Curve and Calculations
(1988 \$)

FENCO FIG. 9-10



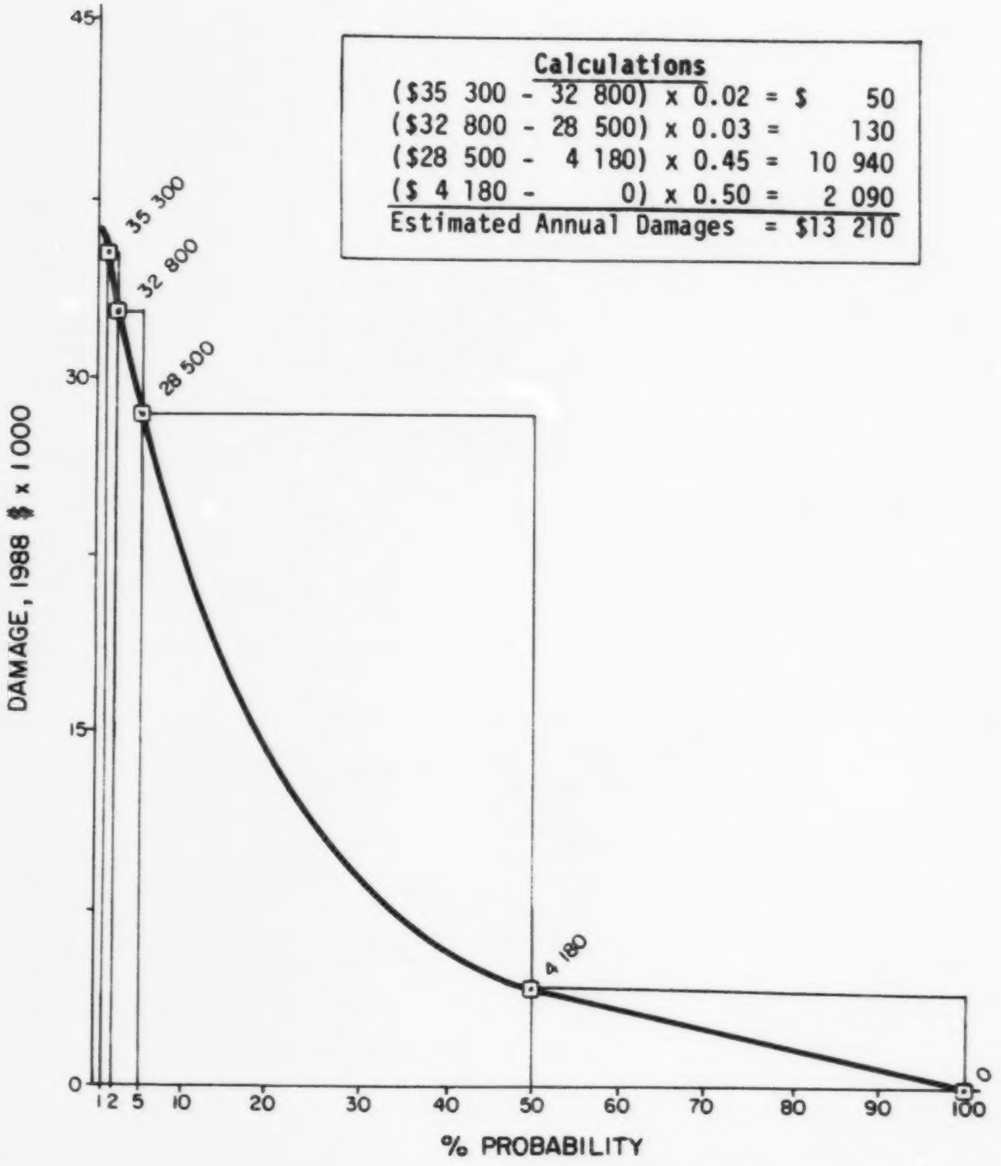
Calculations

($29 335 - 24 625) x 0.02 = $ 90
($24 625 - 19 615) x 0.03 = 150
($19 615 - 5 080) x 0.45 = 6 540
($ 5 080 - 0) x 0.50 = 2 540
Estimated Annual Damages = $9 320

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Lower Dover Street Dikes
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-11



Calculations

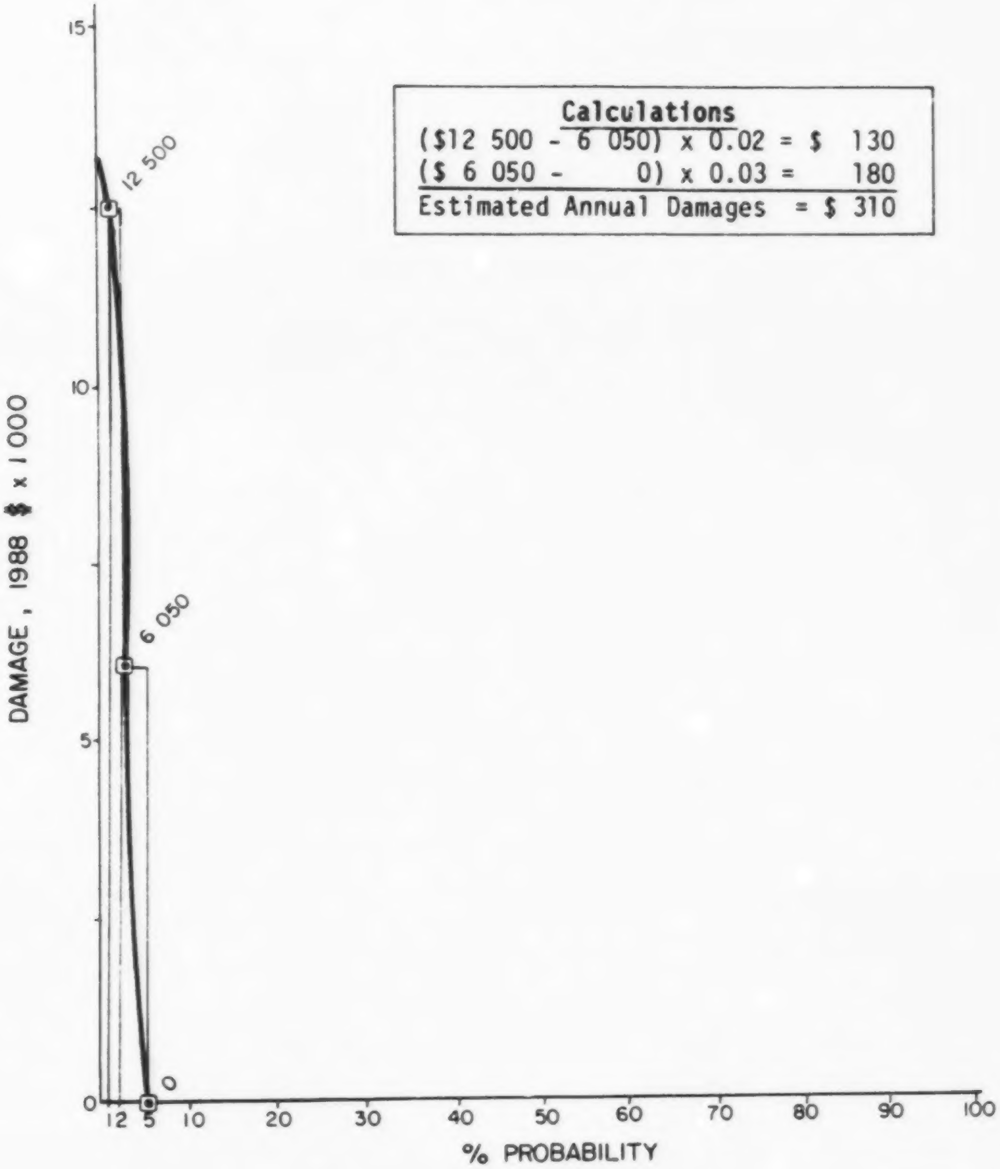
($35 300 - 32 800) x 0.02 = $ 50
($32 800 - 28 500) x 0.03 = 130
($28 500 - 4 180) x 0.45 = 10 940
($ 4 180 - 0) x 0.50 = 2 090

Estimated Annual Damages = $13 210

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Floodproofing
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-12



Calculations

($12 500 - 6 050) x 0.02 = $ 130

($ 6 050 - 0) x 0.03 = 180

Estimated Annual Damages = $ 310

WALKER BROOK
FLOOD PLAIN STUDY PHASE II

Val D'Amour Dike
Estimated Annual Damages Curve and Calculations
(1988 $)

FENCO FIG. 9-13

10.0 CONCLUSIONS & RECOMMENDATIONS

In the same general order in which the text has been presented, the summary conclusions and recommendations follow.

Hydrologic Analyses:

R #1. It is recommended that the peak flow estimates associated with the upper 95% confidence limits from the statistical analyses be adopted for the flood damage reduction process.

C #1. It is concluded that the flood flow values established on the basis of Recommendation #1 (R#1) above are approximately 16% less than flood flow estimates derived in earlier studies, with the current work having the benefit of additional and more current streamflow data for verifying the earlier results of flood flow simulations.

Hydraulic Analyses

The conclusions from these analyses were the developed 1:2, 1:20, 1:50 and 1:100 year return period open water flood levels for existing conditions, from which the subsequent work of estimation of areal flood damages and the development and refinement of flood damage reduction options evolved.

Flood Damages

The conclusions from this chapter were used in the subsequent Benefit-Cost Analyses.

It is concluded that, based on the standard methodology and assumptions used in all Flood Damages analyses under the Program, there is inherently an underestimation of damages to the public property components. As an example: the immediately identifiable flood damages to a bridge structure, as documented at the time of the event or upon completion of the follow-up repairs, are incorporated into the analyses. However, there is no recognition or incorporation of the reduced life costs that may very well occur to the same structure should there be residual deterioration such as long term subsidence.

In some instances, recognition of such longer term flood damages costs could have a substantial affect on the benefit-cost ratios.

Flood Damage Reduction Options

C #2. It was determined that a considerable portion of the out-of-channel spills originating near the Route 11/Route 270 interchange, and affecting the St. Albert area, can be controlled by modest remedial works in the immediate upstream area of the interchange. However, Chapter 7, Engineering & Environmental Considerations, concluded that there were also considerable "spill flows" affecting the St. Albert area that were completely independent of Walker Brook flows.

R #2. It is recommended that the findings relative to the spill flows to the St. Albert area that originate from overland flows from the three adjacent catchment areas to the north and west of the interchange be brought to the attention of the appropriate authorities of the municipalities of Atholville and Campbellton.

C #3. It was concluded that non-structural measures of early warning and contingency planning, floodproofing of structures in the flood-prone areas immediately west of Pine Street and at Dover and Ortona Streets, and zoning of areas within the currently estimated 1:100 year flood plain to restrict and control development, would each be productive measures under the Program.

R #3. It is recommended that the necessary steps be taken to proceed to implementation of the non-structural measures noted in C #3 preceding.

Engineering & Environmental Considerations

C #4. It is concluded that in the Atholville/Campbellton area, Basic Option a), the Route 11 diversion channel, has many more desirable and practical features for successful implementation and flood damage reduction than does the Basic Option b), where all major construction is located in the existing Walker Brook flood plain.

Benefit-Cost Analyses

Further to Conclusion #4 previous, having confirmed that an estimated expenditure of approximately $703 400 will completely eliminate the 1:100 year return period flooding of existing development, and the severance of all transportation to and from in excess of 400 private residences and 25 commercial and public properties in the Atholville/Campbellton area, it is therefore recommended that:

R #4. immediate consideration be given to proceeding with the re-grading, floodwall construction with riprap protection, and installation of twin 1 200 mm diameter concrete culverts in the area of the sharp curve in Route 270 immediately east of the Sugarloaf Park entrance, at an estimated cost of $80 400;

R #5. negotiations be initiated with the Provincial Department of Transportation with a view to developing a floodflow diversion channel along the south side of the existing Route 11, from a controlled flow point of entry at the existing Walker Brook/Route 11 point of crossing to a new crossing of Route 11 some 1.25 km to the east, and hence discharging back into Walker Brook downstream of the currently vulnerable developed area of the flood plain, - at a combined estimated cost of R #4 and R #5 of approximately $703 400, with a benefit-cost ratio of 1.58:1, and an estimated Average Annual Damage Reduction of 100%.

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